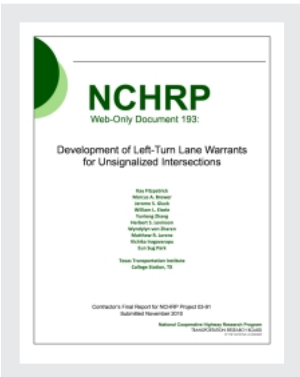

Development of Left-Turn Lane Warrants for Unsignalized Intersections

DETAILS

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AUTHORS

Fitzpatrick, Kay; Brewer, Marcus A.; Eisele, William L.; Levinson, Herbert S.; Gluck, Jerome S.; Lorenz, Matthew R.; Zhang, Yunlong; von Zharen, Wyndylyn; Iragavarapu, Vichika; and Park, Eun Sug

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Kay Fitzpatrick, TTI senior research engineer, was the principal investigator. The other authors of this report are Marcus A. Brewer (assistant research engineer, TTI), Jerome S. Gluck (associate vice president, AECOM), William L. Eisele (research engineer, TTI), Yunlong Zhang (assistant professor, Texas A&M University; assistant research scientist, TTI), Herbert S. Levinson (principal, Herbert S. Levinson, Transportation Consultant), Wyndylyn von Zharen (professor, Texas A&M University at Galveston), Matthew R. Lorenz (senior traffic engineer, AECOM), Vichika Iragavarapu (assistant transportation researcher, TTI), and Eun Sug Park (associate research scientist, TTI). The work was performed under the general supervision of Dr. Fitzpatrick.

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ABSTRACT

Left-turn movements at intersections, including driveways—especially movements that are made from lanes that are shared with through traffic—cause delays and adversely impact safety. Although left-turn warrants have been updated, many agencies still use research performed by M. Harmelink from the mid-1960s. While most states use procedures that are based on Harmelink, a number of limitations of Harmelink's procedure have been identified. Economic analysis can provide a useful method for combining traffic operations and safety benefits of left-turn lanes to identify situations in which left-turn lanes are and are not justified economically. This project used a benefit-cost approach to determine when a left-turn lane would be justified. The steps included simulation to determine delay savings from installing a left-turn lane, crash costs and crash reduction savings determined from safety performance functions and accident modification factors available in the *Highway Safety Manual*, and construction costs. Left-turn lane warrants were developed for rural two-lane highways, rural four-lane highways, and urban and suburban roadways. In addition, warrants for bypass lanes were developed for rural two-lane highways. A *Design Guide on Left-Turn Accommodations at Unsignalized Intersections* was developed that discusses left-turn lane designs, traffic control treatments, and case study examples.

EXECUTIVE SUMMARY

Left-turn movements at intersections, including driveways—especially movements that are made from lanes that are shared with through traffic—cause delays and adversely impact safety. The objectives for this National Cooperative Highway Research Program (NCHRP) research were to develop an objective and clear process for the installation of left-turn lanes at unsignalized intersections and provide guidance on the design of these lanes.

A review of the literature was performed using many sources, including research reports, state and federal design manuals, and handbooks. Although many procedures are currently in use by various organizations to determine the need for left-turn lanes, several are either very similar or identical. Most states' criteria are based on the American Association of State Highway and Transportation Officials (AASHTO) *A Policy on Geometric Design of Highways and Streets* (commonly known as the *Green Book*) values or *NCHRP Report 279* values—both of which are based on work done in the 1960s by M. Harmelink.

Interviews were conducted of representatives from state departments of transportation, county governments, city governments, and consultants. The 25 questions in the interview were structured into planning, design, legal/policy/finance, and potential future applications. All interview participants indicated that left-turn treatments are provided at unsignalized intersections. In addition to left-turn lanes, many use two-way, left-turn lanes and bypass or shoulder widening (typically used when options are limited). One respondent indicated roundabouts are another treatment considered for dealing with left-turn movements.

A legal review conducted as part of this research addressed the following question: When a government seeks to fulfill a broad public objective such as safety and, in this project, left-turn accommodation, who should bear the costs—the developer who would be adding traffic to the roadway network or the general public? Unless the facts of the case are clear cut, the outcome is unpredictable even within specific jurisdictions. Results will be more predictable if agencies have the documented authority to manage access. This authority could be achieved in a number of different ways (state or municipal codes for access management, administrative rules, etc.). Otherwise, there are too many hazy cases and multiple factors so that predicting the outcome is impossible.

This project's driver behavior study used videotaped recordings of vehicle and pedestrian operations collected at 30 sites located in College Station/Bryan, Texas; Houston, Texas; Staten Island, New York; and Phoenix, Arizona. The behaviors for a total of 2945 vehicle drivers that started from a stopped position were used in the evaluation. The most influential variables on the amount of time used to clear the intersection are crossing width and posted speed limit. The relationship between clearance time and the accepted lag or gap time was in the direction expected although it did not have as large an influence as initially thought. Gap is defined as the time interval between two opposing vehicles that is necessary for a left-turning vehicle to safely complete a left-turn maneuver. The relationship between gap and posted speed limits from the field studies was similar to the finding from a simulator study reported in the literature—smaller gaps are accepted at higher posted speed sites. Gap acceptance values increase as the crossing width increases, although only by a small amount (less than 1 sec between the one-lane group and the two-lane or very-wide-one-lane group).

Because of several concerns with the Harmelink procedure, including the lack of a clear relationship between the assumptions in the model and delay or safety on a highway, the research team recommends that the results from the benefit-cost ratio be used as the basis for left-turn lane warrants.

Economic analysis can provide a useful method for combining traffic operations and safety benefits of left-turn lanes to identify situations in which left-turn lanes are and are not justified economically. A benefit-cost approach was used in this project to determine when a left-turn lane would be justified. The steps included simulation to determine delay savings from installing a left-turn lane; crash costs, crash reduction savings determined from safety performance functions and accident modification factors available in the AASHTO *Highway Safety Manual;* and construction costs. For rural conditions, different safety performance functions are provided for two- and four-lane highways and for three- and four-leg intersections. For urban and suburban arterials, prediction equations are provided for three- and four-leg intersections. Separate urban and suburban prediction equations are not provided based on the number of lanes on the major-road approach. The prediction equations are not a function of speed limit; therefore, the developed warrants are also not a function of speed limit.

A range of values was used in the benefit-cost evaluation to identify volume conditions when the installation of a left-turn lane at unsignalized intersections and major driveways would be cost-effective. Plots and tables were developed that indicate combinations of major-road traffic and left-turn lane volume where a left-turn lane would be recommended. Warrants were developed using the following:

- A range of values for the economic value of a statistical life,
- Crash costs based on values in the *Highway Safety Manual*,
- A range of construction costs, and
- A benefit-cost ratio of 1.0 and 2.0.

The research team suggests a benefit-cost ratio of 1.0 along with the mid-range economic value of a statistical life and moderate construction cost to identify the warrants for a left-turn treatment. For urban and suburban areas, that is a left-turn lane. For rural two-lane highways, that is a bypass lane. A benefit-cost ratio of 2.0 has been argued as being a more practical value to use to offset the potential variability in other assumptions. The warrants based on a benefit-cost ratio of 2.0 were selected for a left-turn lane on rural two-lane highways. These values were similar to the warrants that resulted when the lower crash costs based on older *Highway Safety Manual* values were used.

Left-turn lanes can reduce the potential for collisions and improve capacity by removing stopped vehicles from the main travel lane. Left-turn lane warrants were developed as part of NCHRP Project 3-91 using an economic analysis procedure for rural two-lane highways, rural four-lane highways, and urban and suburban roadways. NCHRP Project 3-91 also developed a *Design Guide on Left-Turn Accommodations at Unsignalized Intersections* that discusses left-turn lane designs, traffic control treatments, and case study examples.

CHAPTER 1

INTRODUCTION

BACKGROUND

Left-turn movements at intersections, including driveways—especially movements that are made from lanes that are shared with through traffic—cause delays and adversely impact safety. Although updated warrants have been developed in some jurisdictions for when to provide left-turn lanes, many agencies still use research from the mid-1960s. The research by M. Harmelink in Canada that was published in 1967 focused on "Volume Warrants for Left-Turn Storage Lanes at Unsignalized Grade Intersections" (*1*). As indicated in recent research, some values used in the methodology are no longer valid. In addition, current conditions require a broader assessment of when to provide left-turn accommodations. Explicit consideration of safety and operations (e.g., delay) is needed. Technical warrants are an important element of the decision-making process.

The addition of a left-turn lane can improve the operations and safety at an intersection.

Functions of left-turn lanes include:

- Reduce the number of conflicts and crashes;
- Separate through, turning, and/or queuing traffic;
- Decrease delay and increase capacity;
- Provide more operational flexibility and impacts; and
- Provide an area for left-turning vehicles to decelerate outside of the through traffic lane.

Factors considered when making the decision to install a left-turn lane can include:

- Type/function of roadway,
- Number of lanes,
- Prevailing speeds,
- Traffic control/operations,
- Turn and other volumes,
- Roadway(s) alignment, and
- Safety (conflict, crash numbers, and crash types/causes).

A recent Federal Highway Administration (FHWA) study (*2*) found that the addition of a left-turn lane can result in reductions of crashes from 7 to 48 percent (see Table 1). Other studies (*3*) have demonstrated the benefits of delay reductions with the installation of a left-turn lane on two-lane highways. Guidelines as to when to include a left turn in intersection design are plentiful. Some are based on minimizing conflicts in terms of the occurrence of a through vehicle arriving behind a turning vehicle, some are based on decreasing the amount of delay to through vehicles, and others are based on consideration of safety. Because of the quantity of methods, questions are asked regarding which method to use. For example, are certain techniques better for a rural versus an urban setting? Do the evaluations differ for number of lanes and for type of intersection?

Table 1. Expected percent reduction for adding turn lane identified in FHWA study (*2*).

Treatment/Area/Intersection Type			Expected Percent Reduction of Total Crashes			
			One Approach		Both Approaches	
			Stop	Signal	Stop	Signal
Add Left-Turn Lane	Rural	3-leg	44	15	NA	NA
		4-leg	28	18	48	33
	Urban	3-leg	33	7	NA	NA
		4-leg	27	10	47	19
Add Right-Turn Lane	Rural/Urban	All	14	4	26	8
NA = value not identified in research						

OBJECTIVES

The objectives for this research were to:

- Develop an objective and clear process for the selection of left-turn accommodations at unsignalized intersections and
- Provide guidance on the design of these accommodations.

APPROACH

The research for the project was conducted within tasks split into two phases. Phase I focused on reviewing the literature, conducting interviews, identifying performance measures, identifying sources for data, and developing the Phase II work plan. In Phase II data were collected at 30 sites to be used to calibrate a simulation model and to update the assumptions in the Harmelink approach. An economic analysis through a benefit-cost approach was conducted to identify warrants. The benefit-cost ratio included consideration of crash savings, delay savings, and construction costs. The simulation model was used to determine expected delay reduction when a left-turn lane is added. Crash savings and construction costs were identified as part of this project. Economic analysis provides a useful method for combining traffic operations and safety benefits of left-turn lanes to identify situations in which left-turn lanes are and are not justified.

A *Design Guide* (*4*) was developed as part of this research project. It contains the recommended left-turn lane warrants along with design and traffic control treatment discussions for left-turn lanes.

ORGANIZATION OF THIS REPORT

The research for this project is presented in the following chapters and appendices:

- Chapter 1: Introduction. This chapter provides an overview of the research problem and the approaches used in the research. It also presents the objectives of the research project.
- Chapter 2: Literature Review—Installation Guidelines. A review of the literature was performed using many sources including research reports, state and federal design manuals, and handbooks. Details are provided on those methods that appear to have distinctive results.

- Chapter 3: Literature Review—Left-Turn Lane Design. This chapter contains information on the design of left-turn lanes, reviewing information from state design manuals and national reference documents, as well as recent research.
- Chapter 4: Driver Behavior Study. The methodology used and results from the field study at 30 intersections are documented in this chapter.
- Chapter 5: Delay, Crash, and Construction Cost Studies. A benefit-cost approach was used to determine when a left-turn lane installation is justified. The approach uses the benefits from crash reductions (change in number of crashes) and delay reduction (improvements in roadway capacity from removing the slow-moving or stopped left-turn vehicles) and compares them to the cost of constructing a left-turn lane.
- Chapter 6: Comparison of Procedures. The findings from the research are used to generate suggested warrants using the benefit-cost ratio and the approach developed by Harmelink.
- Chapter 7: Summary and Conclusions. The final chapter of the report provides a summary of the research along with the research team's conclusions.
- Appendix A: Revised Text on Left-Turn Lane Warrants for the AASHTO *Green Book.* This appendix presents the suggested revisions to the AASHTO *Green Book* based on the research.
- Appendix B: Revised Text on Left-Turn Lane Warrants for the TRB *Access Management Manual*. This appendix presents the suggested revisions to the Transportation Research Board (TRB) *Access Management Manual* based on the research.
- Appendix C: State Warrants/Guidelines for Left-Turn Lanes. A summary of the warrants and guidelines for left-turn lanes from states is presented in this appendix.
- Appendix D: Interview Questions. This appendix lists the interview questions. The 25 questions in the interview were structured into planning, design, legal/policy/finance, and potential future applications.
- Appendix E: Interview Findings. Results from the interviews conducted of representatives from state departments of transportation, county governments, city governments, and consultants are documented in this appendix.
- Appendix F: Legal Review. The legal review conducted as part of this research addressed the following question: When a government seeks to fulfill a broad public objective such as safety and, in this project, left-turn accommodation, who should bear the costs—the developer who would be adding traffic to the roadway network or the general public?

CHAPTER 2

LITERATURE REVIEW—INSTALLATION GUIDELINES

A review of the literature was performed using many sources including research reports, state and federal design manuals, and handbooks. Although many procedures are currently in use by various organizations to determine the need for left-turn lanes, several are either very similar or identical. Details are provided below on those methods that appeared to have unique results.

LEFT-TURN LANE INSTALLATION GUIDELINES

Guidelines Based on Risk

Common terms are used in several of the techniques. Figure 1 graphically shows the following movements that are used to determine the need for a left-turn lane in several of the guidelines:

- Advancing volume (V_A)—the total peak hourly volume of traffic on the major road approaching the intersection from the same direction as the left-turn movement under consideration.
- Left-turn volume (V_L)—the portion of the advancing volume that turns left at the intersection.
- Percent left turns (P_L)—the percentage of the advancing volume that turns left; equal to the left-turn volume divided by the advancing volume ($P_L = [V_L \div V_A] \times 100$).
- Straight-through volume (V_S)—the portion of the advancing volume that travels straight through the intersection ($V_L + V_S = V_A$).
- Opposing volume (V_O)—the total peak hourly volume of vehicles opposing the advancing volume.

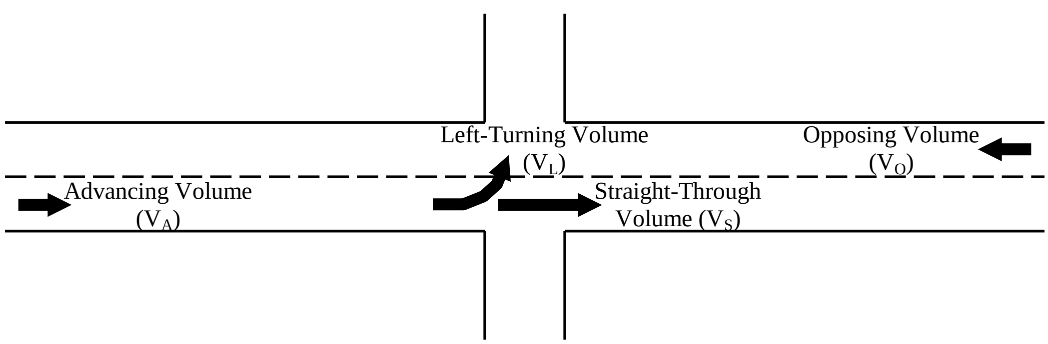
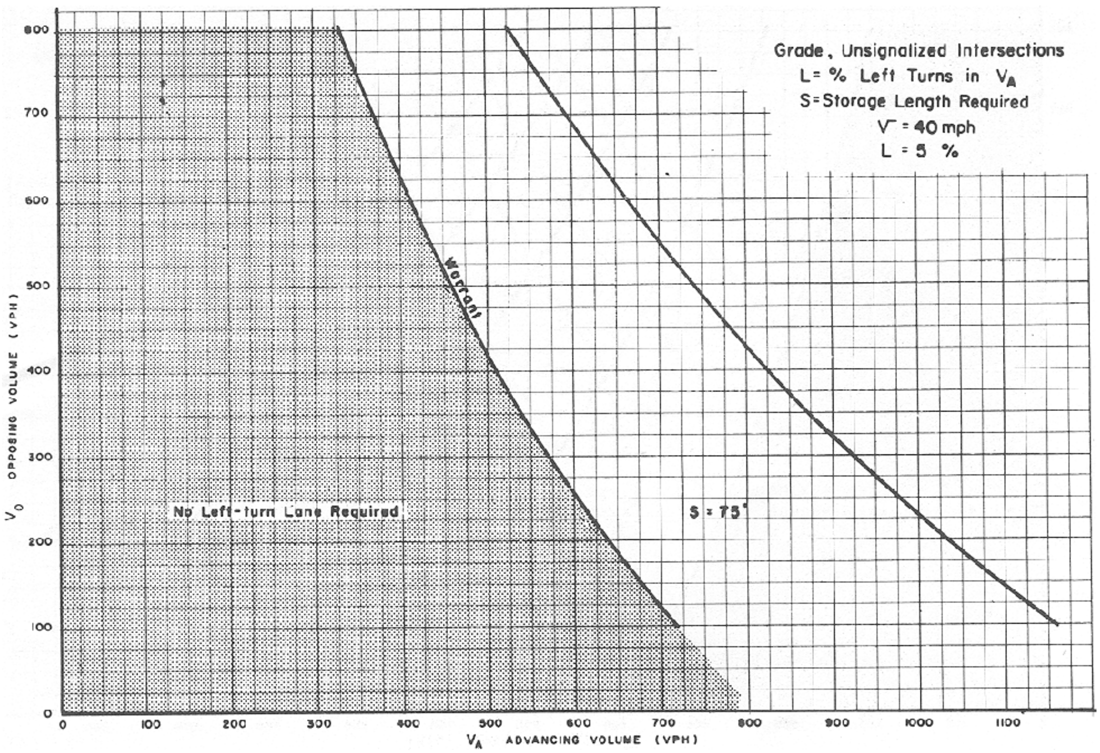


Figure 1. Volumes for use in left-turn lane warrant methods.

Harmelink Method

The oldest research found on evaluating the need for left-turn lanes at unsignalized intersections was that of M. Harmelink (*1*) in a paper that was published in 1967. His research provided the foundation for many current left-turn guidelines. Harmelink based his work on a queuing model in which arrival and service rates are assumed to follow negative exponential distributions. He states that the probability of a through vehicle arriving behind a stopped, left-turning vehicle should not exceed 0.02 for 40 mph, 0.015 for 50 mph, and 0.01 for 60 mph. He presents his criteria in the form of graphs, 18 in all. To use his graphs, the advancing volume, opposing volume, operating speed, and left-turn percentage need to be known. Graphs for speeds of 40, 50, and 60 mph are given, as well as 5, 10, 15, 20, 30, and 40 percent left-turn volumes. An example graph of Harmelink's criteria for determining the need for left-turn lanes is shown in Figure 2.



Source: Harmelink, M., "Volume Warrants for Left-Turn Storage Lanes at Unsignalized Grade Intersections," in *Highway Research Record 211*, Figure 2, p. 9. Copyright, National Academy of Sciences, Washington, D.C., 1967. Reproduced with permission of the Transportation Research Board.

Figure 2. Harmelink (*1*) left-turn lane warrant graph for 40 mph and 5 percent left turns, 1967.

AASHTO

AASHTO's *A Policy on Geometric Design of Highways and Streets* (commonly known as the *Green Book*) (*5*) contains a table for use in determining the need for a left-turn lane on two-lane highways (see Table 2). Similar tables are also present in the 2001 (*6*), 1994 (*7*), 1990 (*8*), and 1984 (*9*) editions of the *Green Book*. The values in the tables are based upon Harmelink's work.

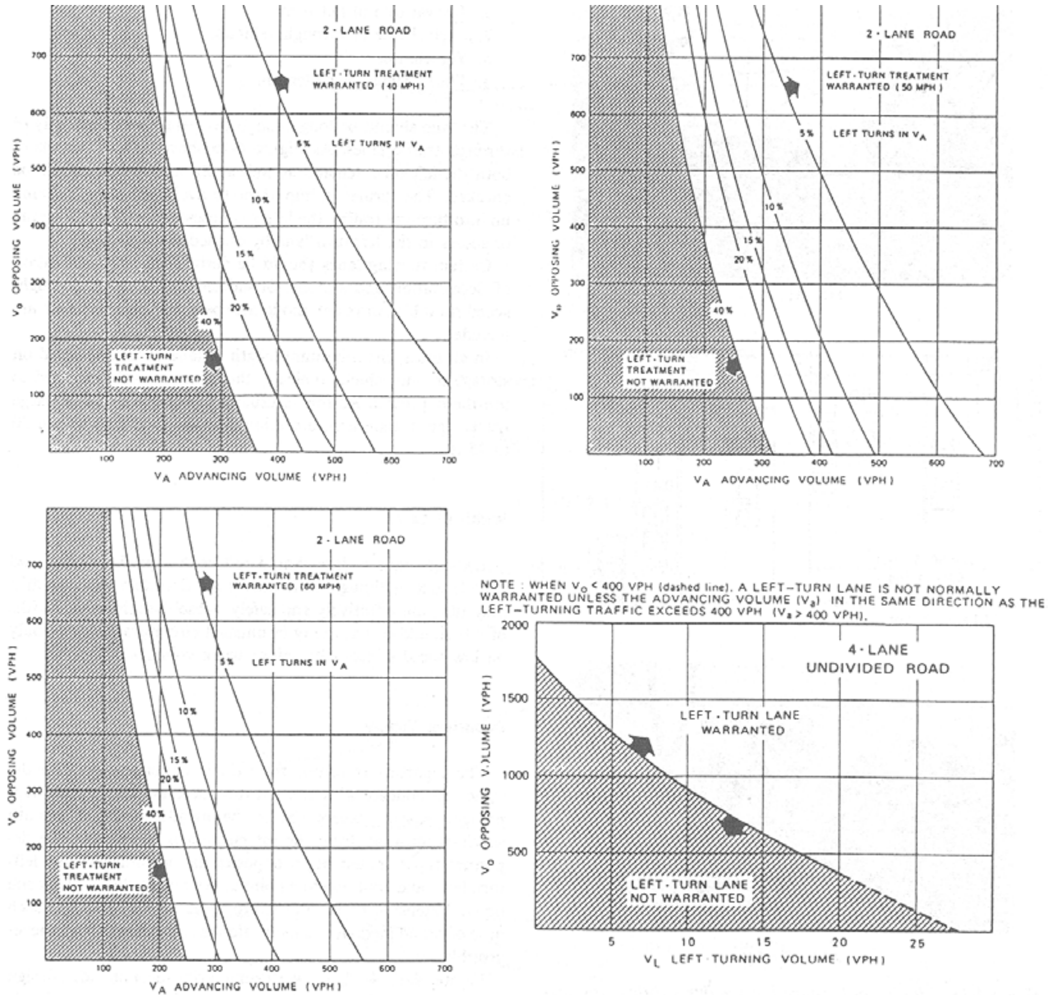
Table 2. AASHTO (5) guide for left-turn lanes on two-lane highways, 2004.

Operating Speed (mph)	Opposing Volume (veh/hr)	Advancing Volume (veh/hr)			
		5% Left Turns	10% Left Turns	20% Left Turns	30% Left Turns
40	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	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	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

NCHRP Report 279

In 1985, the Transportation Research Board published *NCHRP Report 279*, *Intersection Channelization Design Guide (10).* In that report, data from Harmelink's work are used to establish guidelines for determining the need for a left-turn lane. The following advice is provided for unsignalized intersections within new construction:

1. Left-turn lanes should be considered at all median crossovers on divided, high-speed highways.
2. Left-turn lanes should be provided at all unstopped (i.e., through) approaches of primary, high-speed rural highway intersections with other arterials or collectors.
3. Left-turn lanes are recommended at approaches to intersections for which the combination of through, left, and opposing volumes exceeds the warrants shown in Figure 3.
4. Left-turn lanes on stopped or secondary approaches should be provided based on analysis of the capacity and operations of the unsignalized intersection. Considerations include minimizing delays to right-turning or through vehicles and total approach capacity.



Source: Neuman, T., *Intersection Channelization Design Guide, NCHRP Report 279*. Copyright, National Academy of Sciences, Washington, D.C., 1985.

Figure 3. *NCHRP Report 279* (*10*) left-turn lane guidelines, 1985.

NCHRP Report 279 also provides guidance for reconstruction/rehabilitation. The report states:

> Addition of left-turn lanes at existing intersections should be considered if safety or capacity problems occur, or if land-use changes are expected to produce significant shifts in local traffic patterns (such as increases in left-turn demand). Left-turn lanes can often be added within existing street widths by removing parking, narrowing of lanes or a combination of the two.

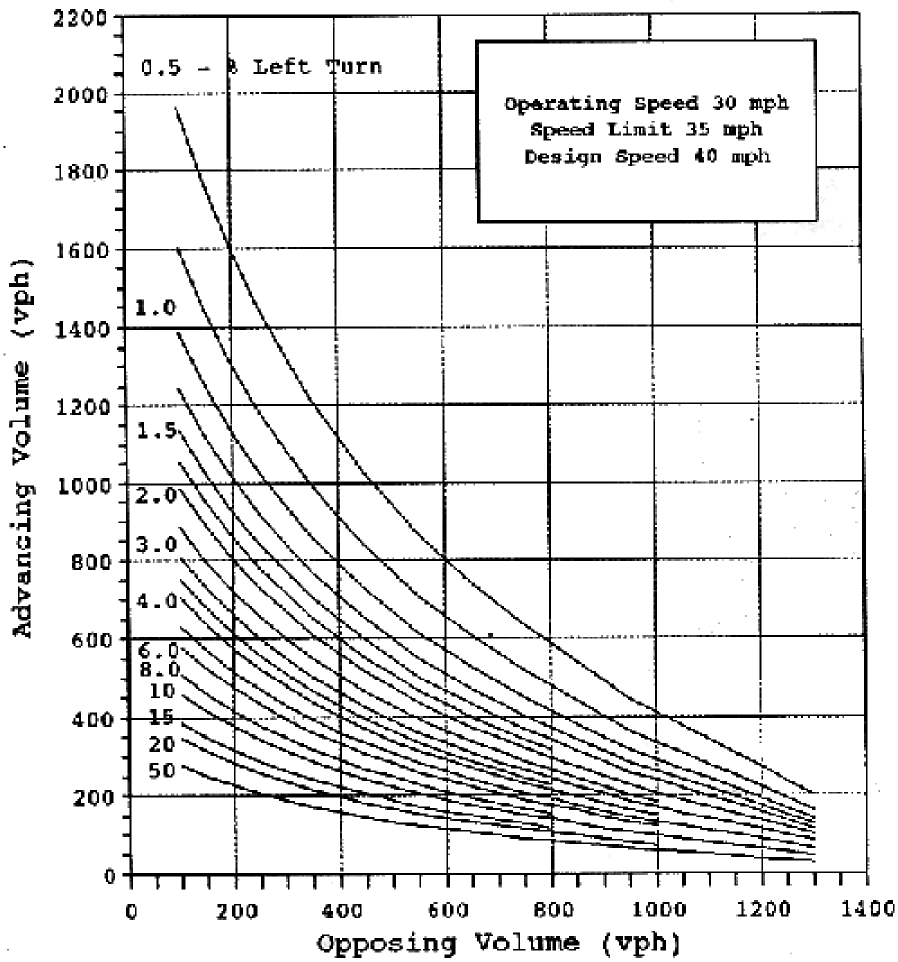
The traffic volume guidelines described for new intersections are also appropriate for evaluating the need for left-turn lanes at existing intersections. In terms of safety, the following guidelines are suggested:

- Left-turn lanes should be considered at intersection approaches that experience a significant number of left-turn-involved (rear-end, left-turn angle, or same direction sideswipe) accidents. A total of four or more such accidents in 12 months, or six or more in 24 months, is considered appropriate.

- When room for separate left-turn lanes is not available, traffic control alternatives should be investigated. Such alternatives to left-turn lane implementation include split phasing at signalized intersections (i.e., operating each approach individually) or prohibition of left turns.

Oppenlander and Bianchi (ITE Technical Committee)

Institute of Transportation Engineers (ITE) Technical Committee 4A-22 (*11*) in the 1980s undertook the task of developing criteria for the provision of separate left-turn lanes at unsignalized and signalized intersections. The work performed by ITE Committee 4A-22 expanded the Harmelink model to include additional speeds (30- and 70-mph roadways) and to include additional left-turn percentages. An example of one of the guideline graphs produced is shown in Figure 4.



Source: Copyright, Institute of Transportation Engineers, Washington, D.C., www.ite.org, 1990. Reproduced with permission.

Figure 4. Oppenlander and Bianchi (*11*) left-turn lane guidelines; unsignalized, two-lane, 30-mph operating speed, 1990.

NCHRP Report 457

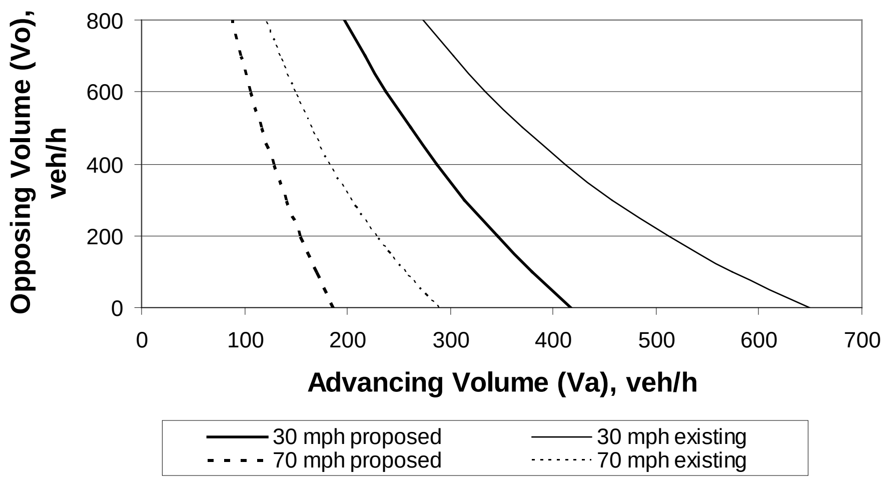
In 2001, Bonneson and Fontaine (*12*) in *NCHRP Report 457* discussed the determination of when to consider a left-turn lane. They cited work by Neuman (*10*) (which was based on the Harmelink model) and re-created the Harmelink model as an interactive spreadsheet (available on the Internet in the NCHRP report at http://trb.org/publications/nchrp/esg/esg.pdf).

Fitzpatrick and Wolff

In 2003, Fitzpatrick and Wolff (*13*) used the following findings from current research in the Harmelink model:

- Critical gap of 5.5 sec (rather than 5.0 sec),
- Time to make a left turn of 4.3 sec (rather than 3.0 sec), and
- Time to clear the lane of 3.2 sec (rather than 1.9 sec).

Table 3 lists the developed suggested guidelines for installing left-turn lanes for operating speeds of 30, 50, and 70 mph. Figure 5 illustrates the changes in the curves for 30 and 70 mph between Fitzpatrick and Wolff and AASHTO.



Source: Fitzpatrick, K., and T. Wolff, "Left-Turn Lane Installation Guidelines," in *2nd Urban Street Symposium*, sponsored by Transportation Research Board, July 2003. Reproduced with permission of the authors.

Figure 5. Fitzpatrick and Wolff (*13*) comparison of existing to proposed guidelines (example uses 10 percent left turns), 2003.

Table 3. Fitzpatrick and Wolff (*13*) guidelines for installing left-turn lanes on two-lane highways, 2003.

Speed (mph)	V_o	Percent Left Turns		
		10	20	40
30	800	197	148	121
	700	217	162	133
	600	238	178	146
	500	261	196	160
	400	286	215	175
	300	314	236	193
	200	345	259	211
	100	380	285	232
	0	418	313	256
50	800	153	115	94
	700	168	126	103
	600	184	138	113
	500	202	152	124
	400	222	166	136
	300	244	183	149
	200	268	201	164
	100	294	221	180
	0	323	243	198
70	800	88	66	54
	700	97	73	59
	600	106	80	65
	500	117	88	71
	400	128	96	78
	300	141	105	86
	200	154	116	95
	100	170	127	104
	0	187	140	114

Van Schalkwyk and Stover

In 2007, Van Schalkwyk and Stover (*14*) discussed additional refinements to the Harmelink curves with a focus on the needs of older drivers. Their paper includes a table of recommended left-turn warrants (see Table 4). They concluded that the left-turn warrants based on Harmelink's 1967 work substantially overestimate the volumes that warrant left-turn lanes. In addition to older driver consideration, they recommended additional research into the differences between positioned and unpositioned drivers.

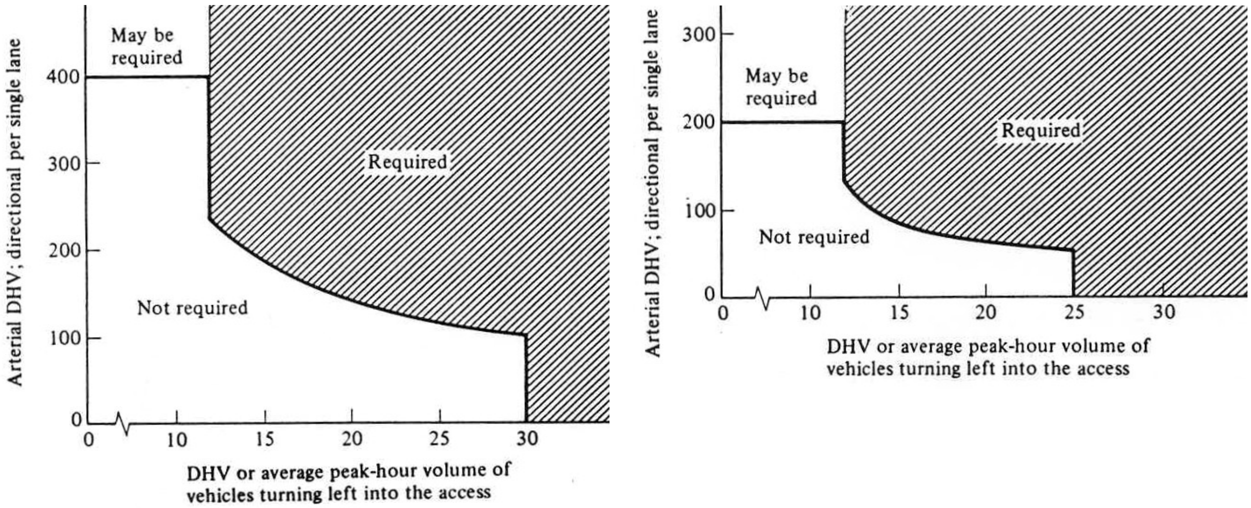
Table 4. Van Schalkwyk and Stover (*14*) recommended left-turn warrants on two-lane highways to accommodate the needs of older drivers, 2007.

Speed (mph)	V_o	Percent Left Turns		
		10	20	40
40	100	214	160	131
	200	189	142	116
	300	167	125	102
	400	147	110	90
	500	129	97	79
	600	113	85	69
	700	99	74	61
	800	87	65	53
50	100	185	139	113
	200	164	123	100
	300	144	108	88
	400	127	95	78
	500	112	84	69
	600	98	74	60
	700	86	64	53
	800	75	56	46
60	100	151	113	93
	200	134	100	82
	300	118	88	72
	400	104	78	64
	500	91	69	56
	600	80	60	49
	700	70	53	43
	800	61	46	38

Guidelines Based on Approaching Volumes

NCHRP Report 348

F.J. Koepke and H.S. Levinson provide two methods for determining the need for left-turn lanes in *NCHRP Report 348* (*15*). The first method is shown in Figure 6; however, Koepke and Levinson state that in most cases, left-turn lanes should be provided where there are more than 12 left turns per peak hour. The second method presents the values included in the *Green Book* for determining whether a left-turn lane should be provided. They also state that “left-turn lanes should be provided when delay caused by left-turning vehicles blocking through vehicles would become a problem.” They emphasize the fact that separate left-turn lanes not only increase intersection capacity but also increase vehicle safety.



(a) 30–35 mph **(b) 40–45 mph**

Source: Koepke, F.J., and H.S. Levinson, *Access Management Guidelines for Activity Centers, NCHRP Report 348.* Copyright, National Academy of Sciences, Washington, D.C., 1992.

Figure 6. *NCHRP Report 348* (*15*) left-turn lane guidelines, 1992.

Guidelines Developed Using Simulation Models

Modur et al.

A 1990s Texas study by Modur et al. (*16*) examined the choice of median design and developed a set of guidelines for determining when to recommend left-turn lanes for arterial streets with speeds less than 45 mph. The guidelines were developed using delay data generated from a simulation model. Table 5 shows the developed guidelines. The authors note that sections with left-turn treatments are better than sections with no treatments, and they recommend that left-turn treatments be used in sections with a disproportionately large number of crashes even though not warranted due to the operational criteria.

Hawley and Stover

Hawley and Stover (*17*) also used delay to generate guidelines on when to install a left-turn lane on four-lane undivided arterials. They considered the delay to through vehicles and asked under what volumes turning vehicles would seriously impact through traffic. They then evaluated the proposed guidelines with a conflict analysis that was based on the probability of two vehicles arriving at the intersection at the same time to assess the safety aspects of the guidelines. A probability of 0.01 was selected as the maximum likelihood of a conflict. The philosophy of the new guidelines focuses on recommending a left-turn lane above a set directional volume rather than a set turn volume.

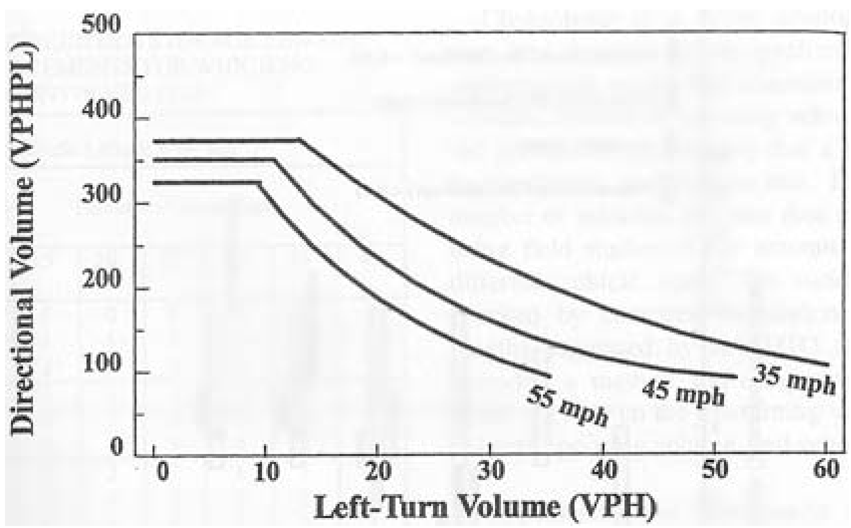
Figure 7 is a graph of the curves recommended. It was developed with consideration of delay (the curved portion of the curves in Figure 7) and probability of a conflict between a left-turning vehicle and advancing vehicles (the horizontal portions of the curves in Figure 7).

Table 5. Modur et al. (*16*) left-turn lane warrant chart, 1990.

Hourly Left-Turn Traffic Volume	Opposing Traffic Volume per Lane per Hour	0–150	150–300	300–450
< 200	400–600	Gray	Black	Black
	200–400	White	Black	Black
	0–200	White	Gray	Gray
200–400	400–600	Black	Black	Black
	200–400	Gray	Black	Black
	0–200	White	Gray	Gray
400–600	400–600	Black	Black	Black
	200–400	Black	Black	Black
	0–200	Gray	Black	Black

Hourly Straight Through Traffic Volume per Lane (columns 0–150, 150–300, 300–450)

Black boxes denote that a left-turn treatment is desirable, provided it can be accommodated within the available right-of-way and pavement width. Gray boxes mean that an operational left-turn treatment may be considered; a left-turn lane or raised median is satisfactory based on individual site considerations. White boxes signify that no left-turn treatment is required based on operational considerations.



Source: Hawley, P., and V. Stover, "Guidelines for Left-Turn Bays at Unsignalized Access Locations," in *Second National Access Management Conference: Conference Proceedings*, Figure 3, p. 388. Copyright, National Academy of Sciences, Washington, D.C., 1996. Reproduced with permission of the Transportation Research Board.

Figure 7. Hawley and Stover (*17*) left-turn lane guidelines for four-lane undivided arterial street with nonplatoon flow, 1996.

Lakkundi et al.

Ranade et al. (*18*) reported on a 2004 study by Lakkundi et al. that developed a set of warrants on the basis of event-based simulation programs that the authors developed and calibrated using six intersections in Virginia. The simulation allowed for stochastic variations in drivers' gap acceptance behavior. Ranade et al. commented that the warrants developed were based on the probability criterion suggested by Harmelink.

Guidelines Based on a Combination of Approaches

Kikuchi and Chakroborty

A 1991 paper (*19*) discusses three approaches to justifying a left-turn lane:

1. Modification to Harmelink's methodology,
2. Delay (average delay to the "caught" through vehicles, average delay to all through vehicles, and delay savings due to the left-turn lane), and
3. Degradation of level of service.

The delay values were determined using simulation. Figure 8 shows an example of the criteria for 10 percent left turns using the three approaches. Other assumptions used in developing the figure include:

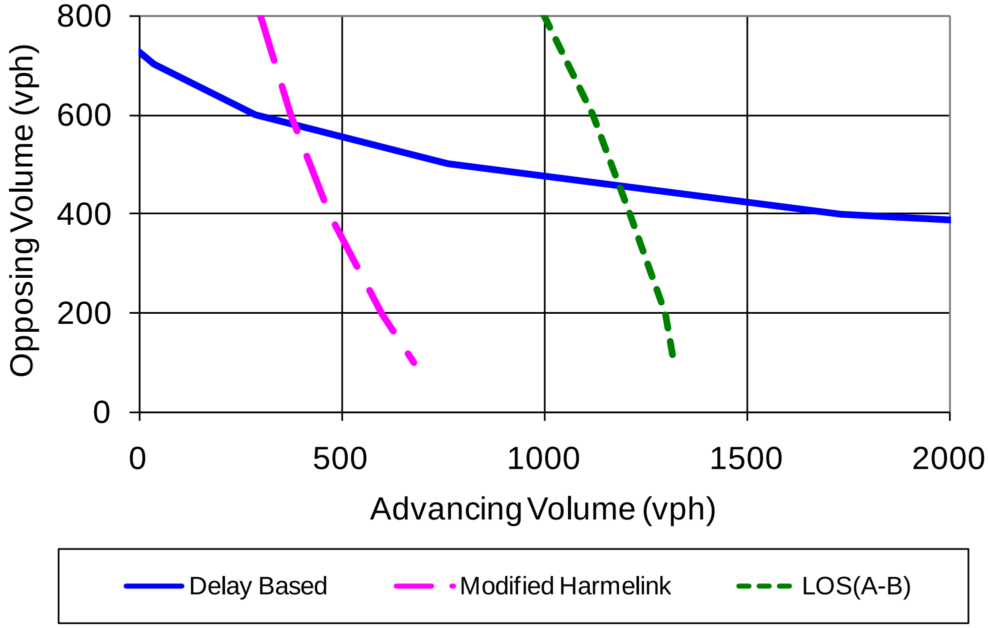
- The probability used in Harmelink's methodology is 0.02.
- The average delay to the through vehicles caught in the queue was assumed to be 19 sec.
- The level of service (LOS) changes from A to B at a through movement capacity of 1800 veh/hr.

The authors support the guidelines developed based on delay or based on level of service because delay and level of service are easier to understand than probability. They note that the precise limits should vary based on the standards of the community and other factors such as the accident experience and the number of buses included in the through vehicles.

Ranade et al.

A 2007 paper (*18*) presents the concept of using an estimate of the benefits of left-turn lane installations at unsignalized intersections as the method to determine when to warrant a left-turn lane. The authors developed a refined decision support system (DSS) for assessing the likely benefits of left-turn lane installations as an aid to deciding whether a left-turn lane is warranted. The developed DSS was designed to predict likely benefits based on:

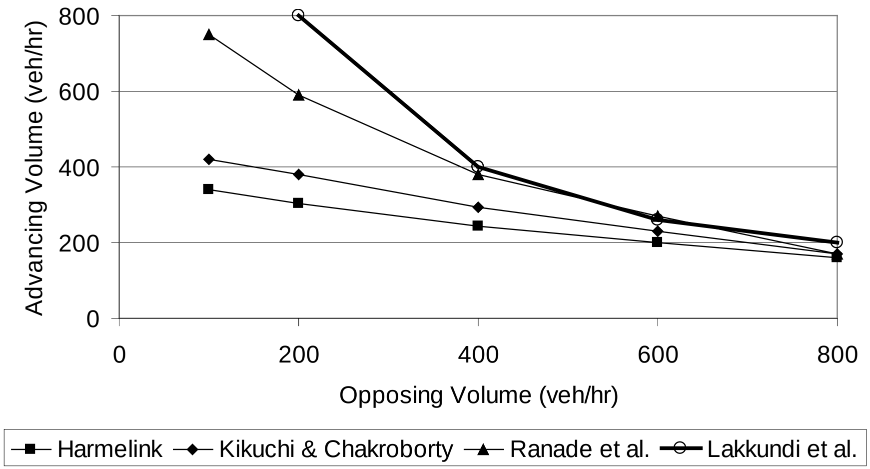
- Delay savings,
- Reductions in percent stops,
- Increases in fuel efficiency, and/or
- Reductions in emissions.



Source: Kikuchi, S., and P. Chakroborty, "Analysis of Left-Turn-Lane Warrants at Unsignalized T-Intersections on Two-Lane Roadways," in *Transportation Research Record 1327*, Figure 7, p. 88. Copyright, National Academy of Sciences, Washington, D.C., 1991. Reproduced with permission of the Transportation Research Board.

Figure 8. Kikuchi and Chakroborty (*19*) left-turn lane guidelines using three methods, 1991.

The first step in developing the DSS was to use microscopic simulation to model several real-world unsignalized intersections with different geometric configurations and located in different area types. After calibrating these models, several scenarios covering a wide range of operational conditions were simulated. The output from these simulation runs was then used to train a set of multilayer perceptron neural networks (NNs). The NNs can then be incorporated into a DSS that can be used to quantify the impacts of a proposed new development as well as estimate the benefits of left-turn lane installations. The authors used the NNs to develop warrants for left-turn lane installations similar to those proposed by Harmelink. Their illustration used a two-lane urban road with an operating speed of 40 mph and 30 percent left turns. They based their case study warrants on the criterion of the total number of stops in the advancing stream. Compared to the original and modified Harmelink models (Kikuchi and Chakroborty), the newly developed warrants allow higher volume combinations before recommending a left-turn lane (see Figure 9).



Source: Ranade, S., A. Sadek, and J. Ivan, "Decision Support System for Predicting Benefits of Left-Turn Lanes at Unsignalized Intersections," in *Transportation Research Record 2023*, Figure 7, p. 36. Copyright, National Academy of Sciences, Washington, D.C., 2007. Reproduced with permission of the Transportation Research Board.

Figure 9. Ranade et al. (*18*) and previous studies (*1, 19*) comparison of left-turn lane warrants (assuming 40-mph speed and 30 percent left turns), 2007.

Summary of Guidelines from Literature

A summary of the findings from the literature can be placed into two major groups—those based on the Harmelink procedure and those based on other procedures. Table 6 summarizes guidelines on warranting a left-turn lane based on the Harmelink procedure. Table 7 summarizes other procedures for warranting left-turn lanes.

Guidelines in State Manuals

Several state manuals also include information on when to consider a left-turn lane. Table 8 summarizes the findings.

Table 6. Summary of selected left-turn lane literature guidelines based on Harmelink procedure.

Method	**AASHTO (*5, 7, 8, 9*)**	***NCHRP Report 279* (*10*), *NCHRP Report 457* (*12*)**	**Oppenlander and Bianchi (*11*)**	**Fitzpatrick and Wolff (*13*)**	**Van Schalkwyk and Stover (*14*)**
Year(s)	**2001, 1994, 1990, 1984**	**1985, 2001**	**1990**	**2003**	**2007**
Roadway Type	Two-lane	Two-way stop controlled	Two-lane unsignalized	Two-lane unsignalized	Two-lane unsignalized
Developed with Consideration of:	Minimize conflict	Minimize conflict	Minimize conflict and increase safety	Minimize conflict	Minimize conflict
Key Feature	Based on Harmelink's 1967 study; developed table of values for various speeds and left-turn percentages	Based on Harmelink's 1967 study; *NCHRP Report 457* includes a spreadsheet to perform calculations	Used Harmelink's model and expanded to additional speed ranges; also added consideration of crashes	Updated variables used in Harmelink's model	Updated variables used in Harmelink's model
Crashes	"…safety considerations are sufficient to warrant them."	States that there are benefits in crash reduction when left-turn lane is added	Crashes by approach that would involve a left-turning vehicle: 4 per year at unsignalized and 5 per year at signalized	Not considered	Not considered

Table 7. Summary of selected left-turn lane literature guidelines based on approaching volume, delay, or combination of approaches.

Method	***NCHRP Report 348* (*15*)**	**Modur et al. (*16*)**	**Hawley and Stover (*17*)**	**Kikuchi and Chakroborty (*19*)**	**Ranade et al. (*18*)**
Year(s)	**1992**	**1990**	**1996**	**1991**	**2007**
Roadway Type	Any unsignalized	Urban (roadways less than 45 mph)	Four-lane undivided	Two-lane unsignalized	Two-lane unsignalized
Developed with Consideration of:	Not specified	Delay	Delay	3 approaches: • Modify Harmelink • Delay • Degradation of LOS	Benefits based on: • Delay • Reduction in percentage of stops • Increase in fuel efficiency • Reduction in emissions
Key Feature	Would recommend lower left-turn volumes than other methods	Used simulation to determine guidelines	Used results from simulation to determine value	Consider delay or level of service easier to understand than probability	Simulation used to train a set of multilayer perception neural networks
Crashes	"Separate turning lane …promote the safety of all traffic."	"Sections with left-turn treatments are always better than the sections with no treatment."	Guidelines checked against maximum probability of conflict of 0.01; recommends consideration of potential crashes	Accident experience should be considered	Not considered

Table 8. Summary of state methods.

	Primary Procedure							Consideration of:							
State	***Green Book***	**279 Graphs**	**279 Guidance**	**General Guidance**	**Unique Guidance**	**Median Openings**	**Specific Office**[1]	**Crashes/Safety**	**Classification**	**Consistency**	**Sight Distance**	**Capacity Eval.**	**Cost**	**Catalyst**[2]	**Land Use**
Alaska	✓			✓		✓									
Arizona							✓								
California							✓								
Colorado			✓			✓			✓			✓			
Connecticut		✓	✓			✓		✓	✓		✓	✓			
Delaware	✓		✓			✓		✓	✓		✓	✓			
Florida				✓				✓							
Georgia					✓			✓					✓	✓	
Illinois		✓	✓					✓	✓	✓	✓	✓			
Indiana	✓			✓	✓	✓		✓	✓	✓	✓	✓			
Iowa					✓										✓
Kentucky			✓			✓					✓				
Louisiana				✓		✓		✓	✓		✓	✓			
Massachusetts	✓							✓			✓				✓
Minnesota					✓	✓	✓	✓	✓		✓	✓	✓	✓	✓
Mississippi		✓	✓			✓		✓			✓	✓			
Missouri		✓						✓							
Montana		✓	✓			✓		✓			✓	✓			
Nebraska		✓	✓			✓		✓							
Nevada	✓				✓										
New Jersey	✓			✓											
New Mexico					✓										
New York	✓							✓			✓	✓	✓		
Ohio					✓							✓	✓		
Oregon					✓	✓		✓			✓				
Pennsylvania				✓											
South Dakota					✓	✓		✓			✓				
Tennessee	✓	✓													
Texas	✓							✓							
Utah					✓				✓			✓			
Virginia	✓					✓						✓			
Washington	✓		✓					✓				✓			

[1] Decision is made within a specific office within the department of transportation.
[2] Criteria may vary by catalyst that triggers consideration of left-turn accommodation: major reconstruction (or new construction), spot improvement, or development.

Based on Green Book

The following nine state manuals either include the same table of criteria as the values included in the *Green Book* (*5*) for determining the need for a left-turn lane or reference the *Green Book*:

- *Alaska Highway Preconstruction Manual* (*20*),
- Delaware's *DelDOT Road Design Manual* (*21*),
- *Indiana Design Manual* (*22*),
- *Nevada Access Management System and Standards* (*23*),
- *The Commonwealth of Massachusetts Highway Design Guidebook* (*24*),
- *New York Highway Design Manual* (*25*),
- *Texas Roadway Design Manual (26),*
- *Virginia Road Design Manual* (*27*), and
- *Washington Design Manual* (*28*).

Three of these states also included information from *NCHRP Report 279* (*10*) or from Harmelink's original paper (*1*).

These nine states also typically provided general guidelines regarding left-turn lanes. For example, the *Indiana Design Manual* (*22*) states:

> The accommodation of left turns is often the critical factor in proper intersection and median-opening design. A left-turn lane can significantly improve both the level of service and intersection safety. An exclusive left-turn lane should be provided as follows:
>
> - At each intersection on an arterial, where practical;
> - At each intersection on a divided urban or rural highway with a median wide enough to accommodate a left-turn lane, provided that adequate spacing exists between intersections;
> - At an unsignalized intersection on a two-lane urban or rural highway which satisfies the criteria shown in [a table that contains the *Green Book* values];
> - At an intersection where a capacity analysis determines that a left-turn lane is necessary to meet the level-of-service criteria, including multiple left-turn lanes;
> - At a signalized intersection where the design-hour left-turning volume is 60 veh/h or more for a single turn lane, or where capacity analysis determines the need for a left-turn lane;
> - For uniformity of intersection design along the highway if other intersections have left-turn lanes in order to satisfy driver expectancy;
> - At an intersection where the accident experience, traffic operations, sight distance restrictions (e.g., intersection beyond a crest vertical curve), or engineering judgment indicates a significant conflict related to left-turning vehicles; or
> - At a median opening where there is a high volume of left turns, or where vehicular speeds are 50 mph or higher.

Delaware's *DelDOT Road Design Manual* (*21*) states that for unsignalized intersections, left-turn lanes should be provided:

- At all median openings on high-speed divided highways,
- On approaches where sight distance is limited,
- At non-stopping approaches of rural arterials and collectors, and
- At other approaches where required based on capacity and operational analysis.

The manual also notes that there may be other needs, primarily safety, for left-turn lanes at other locations than mentioned in the general guidelines. For two-lane roadways, a high-volume, intersecting minor roadway or entrance may also create the need to separate movements with auxiliary turning lanes. The manual includes tabulated values similar to the values in the *Green Book.*

The *Indiana Design Manual* (*22*) provides information on warrants for passing blisters. Passing blisters are used to relieve congestion due to left-turning vehicles. Appendix B includes a copy of a layout for a passing blister. Indiana states the following is to be reviewed to determine the need for a passing blister:

- Traffic volume. A passing blister may be provided at the intersection of a public road or street with a two-lane state highway with a design-year AADT [annual average daily traffic] of 5000 or greater. For a two-lane state highway with a design-year of less than 5000, a passing blister should be used only if one or more of the following occurs:
    - There is an existing passing blister.
    - There are 20 or more left-turning vehicles during the design hour.
    - Accident reports or site evidence, such as skid marks in the through lane displaying emergency braking, indicate potential problems with left-turning vehicles.
    - The shoulder indicates heavy use (e.g., dropped shoulder, severe pavement distress).
- The decision on whether to use either a channelized left-turn lane or a passing blister should be based on accident history, right-of-way availability, through- and turning-traffic volumes, design speed, and available sight distance. A channelized left-turn lane should be provided if the left-turning volume is high enough that a left-turn lane is warranted.

Based on NCHRP Report 279

The state manuals that include the graphs available in *NCHRP Report 279* along with some of the recommendations are:

- Connecticut's *ConnDOT Highway Design Manual* (*29*),
- Illinois' *Bureau of Design and Environment Manual* (*30*),
- Mississippi's *Roadway Design Manual (31),*
- *Missouri Engineering Policy Guide* (*32*),
- Montana's *Road Design Manual* (*33*), and
- Tennessee's *TDOT Roadway Design Guidelines* (*34*).

For example, the Connecticut manual (*29*) states:

> In general, exclusive left-turn lanes should be provided for at-grade intersections as follows:
>
> 1. on all divided urban and rural highways with a median wide enough to allow a left-turn lane (this applies to intersections with public roads and to major traffic generators);
> 2. for all approaches at arterial/arterial intersection;
> 3. at any unsignalized intersection on a two-lane urban or rural highway which satisfies the criteria in Figures 11-5B, 11-5C, 11-5D, 11-5E or 11-5F [These figures are based on *NCHRP Report 279*, which is based on Harmelink. The figures also include graphs for 55 and 45 mph, which were not included in *NCHRP Report 279*. The Connecticut manual does not include the four-lane undivided graph.];
> 4. at any intersection where a capacity analysis determines a left-turn lane is necessary to meet the level-of-service criteria;
> 5. at any intersection included or expected to be within an interconnected signal system where the presence of standing left-turning vehicles would disrupt the progression of platoon traffic; or
> 6. at any intersection where the crash experience, traffic operations, sight distance restrictions or engineering judgment indicates a significant problem related to left-turning vehicles.

The Missouri (*32*) and Tennessee (*34*) manuals also include the *NCHRP Report 279* graphs, but with different descriptive text. The Tennessee manual includes a reference to the *Green Book*.

The Nebraska manual (*35*) refers the reader to the NCHRP report.

The following states include most of the recommendations from *NCHRP Report 279*, without referencing the graphs:

- Colorado (*36*) and
- Kentucky (*37*).

For example, the *Colorado Design Guide* (*36*) includes the following guidelines to facilitate flow where the intersection is unsignalized:

- Left-turn lanes should be considered at all median crossovers on divided, high-speed highways.
- Left-turn lanes should be provided at all uncontrolled approaches of primary, high-speed rural highway intersections with other arterials and collectors.
- Left-turn lanes should be provided on stopped or secondary approaches based on analysis of the capacity and operations of the unsignalized intersection.

Additional material on when to consider deceleration or acceleration lanes is included in Colorado's State Highway Access Code, Volume 2, Code of Colorado Regulations 601-1 (*38*). Criteria are provided by road class. Table 9 lists the criteria.

Table 9. Colorado Access Code (*38*) criteria for deceleration lanes.

Category of Road	Criteria
Expressway, Major Bypass	A left-turn deceleration lane is required for any access with a projected average daily left-turn ingress volume greater than 10. The transition taper length will be included within the required deceleration length. If the projected peak-hour left-ingress turning volume is greater than 10 veh/hr, a left-turn lane with deceleration, storage, and transition taper lengths is required for any access.
Regional Highway	A left-turn deceleration lane with taper and storage length is required for any access with a projected peak-hour left-ingress turning volume greater than 10 veh/hr. The taper length will be included within the required deceleration length.
Rural Highway	A left-turn deceleration lane with taper and additional storage length is required for any access with a projected peak-hour left-ingress turning volume greater than 10 veh/hr. The taper length shall be included within the required deceleration length.
Non-rural Principal Highways	A left-turn deceleration lane and taper with storage length is required for any access with a projected peak-hour ingress turning volume greater than 10 veh/hr. The taper length will be included within the required deceleration length.
Non-rural Arterial	A left-turn lane with storage length plus taper length is required for any access with a projected peak-hour left-ingress turning volume greater than 25 veh/hr. If the posted speed is greater than 40 mph, a deceleration lane and taper are required for any access with a projected peak-hour left-ingress turning volume greater than 10 veh/hr. The taper length will be included within the deceleration length.
Frontage Road	A left-turn lane with storage length plus taper length is required for any access with a projected peak-hour left-ingress turning volume greater than 25 veh/hr. If the posted speed is greater than 40 mph, a deceleration lane and taper are required for any access with a projected peak-hour left-ingress turning volume greater than 10 veh/hr. The taper length will be included within the deceleration length

The Ohio *Location and Design Manual* (*39*) Section 400 (Intersection Design) states:

> Probably the single item having the most influence on intersection operation is the treatment of left turn vehicles. Left-turn lanes are generally desirable at most intersections. However, cost and space requirements do not permit their inclusion in all situations. Intersection capacity analysis procedures of the current edition of the Highway Capacity Manual should be used to determine the number and use of left-turn lanes. For unsignalized intersections, left-turn lanes may also be needed if they meet warrants as provided in [figures that appear to be based on *Green Book* and *NCHRP Report 279* information]. The warrants apply only to the free-flow approach of the unsignalized intersection.

Based on Harmelink

The *New Jersey Roadway Design Manual Section 6: At-Grade Intersections* (*40*) includes general guidance comments. The *New Jersey Administrative Code* (*41*) references the Harmelink paper directly:

> A left-turn lane shall be provided for access points on State highway segments with access level 4 when the criteria set forth in…Highway Research Record 211, Volume Warrants for Left-Turn Storage Lanes at Unsignalized Grade Intersections, incorporated herein by reference, are met. Left-turn access shall be prohibited if the criteria have been met but there is insufficient space for a left-turn lane, unless the Commissioner determines that left-turns can be made safely, considering traffic volumes and sight distances.

General Guidance

General guidance is provided in publications from:

- *Florida Driveway Information Guide* (*42*),
- Louisiana's *Roadway Design Procedures and Details* (*43*),
- *New Jersey Roadway Design Manual Section 6: At-Grade Intersections* (*40*), and
- Pennsylvania's *Design Manual*, Part 2 Highway Design (*44*).

For example, Louisiana's *Roadway Design Procedures and Details* states: "On new four-lane highways, left-turn lanes are usually provided at all intersecting side roads that have dedicated right-of-way." The *New Jersey Roadway Design Manual Section 6: At-Grade Intersections* states: "Median lanes may be provided at intersections and other median openings where there is a high volume of left-turns, or where vehicular speeds are high on the main roadway." Chapter 1 of Pennsylvania's *Design Manual*, Part 2 Highway Design, states: "The warrants for the use of speed-change lanes cannot be stated definitely. However, based on observations and past experience, the following general conclusions have been made: 1. Speed-change lanes are warranted on high-speed and on high-volume highways where a change in speed is necessary for vehicles entering or leaving the through-traffic lanes." The *Florida Driveway Information Guide* states that whenever a driveway is directly served by a median opening, a left-turn lane should be available. This provides for the safest left turns into the driveway. For two-lane roadways, exclusive left-turn lanes should be considered at any location serving the public, especially on curves and where speeds are 45 mph and higher.

Unique Criteria

The following states have unique criteria or procedures for determining when a left-turn lane should be considered:

- Georgia (*45*),
- New Mexico (*46*),
- Iowa (*47*),
- Nevada (*23*),
- Minnesota (*48*, *52*),
- Oregon (*49*),
- South Dakota (*50*), and
- Utah (*51*).

Utah's *Roadway Design Manual of Instruction* (*51*) states: "Under conditions of relatively high traffic volumes, traffic congestion problems can be significantly alleviated with auxiliary lanes to handle turning movements. In rural areas, consider left-turn lanes where there are 25 or more left-turn movements from the main highway in the peak hour."

Georgia's *Regulations for Driveway and Encroachment Control* (*45*) states:

> Left-turn lanes must be constructed at no cost to the Department if the daily site generated left-turn volumes (LTV) based on ITE Trip Generation (assuming a reasonable distribution of entry volumes) meet or exceed the values shown in Table [10] Condition 1. If the LTVs are below the requirements for Condition 1,

the applicant may be required to construct a Right Hand Passing Lane if they meet the criteria in Table [11] Condition 2. The District Access Management Engineer will use engineering judgment to determine if the field conditions would allow construction of the Right Hand Passing Lane [see Appendix B]. Passing lane sections fall under the criteria for two or more lanes.

In the event the District Access Management Engineer determines that field conditions or other factors indicate that it would be in the best interest of the Department to waive the left-turn lane requirement, the District Access Management Engineer must document the recommendations using the form in Appendix E [of the Georgia Manual]. The recommendations shall be approved by the District Engineer and be attached to the Permit. The District Access Management Engineer may also require the addition of a Left-turn lane, even when the conditions in Table [9] are not met, if roadway geometry or field conditions indicate that the safety of the traveling public would be improved. The recommendation must be documented and approved by the District Engineer for inclusion with the Permit.

Table 10. Georgia (*45*) regulations for driveway and encroachment control left-turn requirements, condition 1.

Condition 1: Left-Turn Requirements—Full Construction				
Posted Speed	**2 Lane Routes**		**More than 2 Lanes on Main Road**	
	< 6000 ADT	**≥ 6000 ADT**	**< 10,000 ADT**	**≥ 10,000 ADT**
35 mph or less	300 LTV a day	200 LTV a day	400 LTV a day	300 LTV a day
40 to 50 mph	250 LTV a day	175 LTV a day	325 LTV a day	250 LTV a day
≥ 55 mph	200 LTV a day	150 LTV a day	250 LTV a day	200 LTV a day
ADT = average daily traffic LTV = left-turn volume				

Table 11. Georgia (*45*) regulations for driveway and encroachment control left-turn requirements, condition 2.

Condition 2: Left-Turn Requirements with Right-Hand Passing Lane Option		
Posted Speed	**2 Lane Routes Only**	
	< 4000 ADT	**≥ 4000 ADT**
35 mph or less	200 LTV a day	125 LTV a day
40 to 50 mph	100 LTV a day	75 LTV a day
≥ 55 mph	75 LTV a day	50 LTV a day
ADT = average daily traffic LTV = left-turn volume		

New Mexico's *State Access Management Manual* (*46*) contains details on criteria for left-turn lanes. The information is shown in the following tables:

- Table 12 for urban two-lane highways,
- Table 13 for urban multilane highways,
- Table 14 for rural two-lane highways, and

- Table 15 for rural multilane highways.

Table 12. New Mexico (*46*) criteria for deceleration lanes on urban two-lane highways.

Table 17.B-1. Criteria for Deceleration Lanes on Urban Two-Lane Highways						
Turning Volume[1] (veh/hr)	**Left-Turn Deceleration Lane**			**Right-Turn Deceleration Lane**		
	Minimum Directional Volume in the Through Lane (veh/hr/ln)[2]			**Minimum Directional Volume in the Through Lane (veh/hr/ln)[2]**		
	≤ 30 mph	**35–45 mph**	**45–55 mph**	**≤ 30 mph**	**35–40 mph**	**45–55 mph**
< 5	Not required	Not required	Not required	Not required	Not required	Not required
5	510	450	330	1080	610	360
10	390	330	210	700	400	240
15	320	250	150	500	280	170
20	270	200	120	380	210	140
25	230	160	100	300	180	120
30	200	130	Required	250	160	110
35	170	110	Required	220	150	100
40	150	Required	Required	200	140	Required
45	130	Required	Required	190	Required	Required
≥ 46	Required	Required	Required	Required	Required	Required
	Left-turn deceleration lanes are required on urban two-lane highways for the following left-turn volumes: • ≤ 30 mph: 46 veh/hr or more • 35 to 40 mph: 36 veh/hr or more • 45 to 55 mph: 26 veh/hr or more			Right-turn deceleration lanes are required on urban two-lane highways for the following right-turn volumes: • ≤ 30 mph: 46 veh/hr or more • 35 to 40 mph: 41 veh/hr or more • 45 to 55 mph: 36 veh/hr or more		
Notes: *1. Use linear interpolation for turning volumes between 5 and 45 veh/hr.* *2. The directional volume in the through lane includes through vehicles and turning vehicles.*						

Table 13. New Mexico (*46*) criteria for deceleration lanes on urban multilane highways.

Turning Volume[1] (veh/hr)	Left-Turn Deceleration Lane			Right-Turn Deceleration Lane		
	Minimum Directional Volume in the Adjacent Through Lane (veh/hr/ln)[2]			Minimum Directional Volume in the Adjacent Through Lane (veh/hr/ln)[2]		
	≤ 30 mph	35–45 mph	45–55 mph	≤ 30 mph	35–40 mph	45–55 mph
< 5	**Not Required**	**Not Required**	**Not Required**	**Not Required**	**Not Required**	**Not Required**
5	Not required	490	420	1200	730	450
10	420	370	300	820	490	320
15	360	290	220	600	350	240
20	310	230	160	460	260	180
25	270	190	130	360	230	150
30	240	160	110	290	200	130
35	210	130	100	260	180	120
40	180	120	Required	240	170	110
45	160	110	Required	220	160	Required
≥ 46	140	Required	Required	200	Required	Required
	Left-turn deceleration lanes are required on urban multilane highways for the following left-turn volumes: • ≤ 30 mph: 56 veh/hr or more • 35 to 40 mph: 46 veh/hr or more • 45 to 55 mph: 36 veh/hr or more			Right-turn deceleration lanes are required on urban multilane highways for the following right-turn volumes: • ≤ 30 mph: 56 veh/hr or more • 35 to 40 mph: 46 veh/hr or more • 45 to 55 mph: 41 veh/hr or more		

Notes:
1. *Use linear interpolation for turning volumes between 5 and 55 veh/hr.*
2. *The directional volume in the adjacent through lane includes through vehicles and turning vehicles.*

Table 14. New Mexico (*46*) criteria for deceleration lanes on rural two-lane highways.

Left-Turn Volume[1] (veh/hr)	Left-Turn Deceleration Lane			
	Minimum Directional Volume in Through Lane (veh/hr/ln)[2]			
	≤ 30 mph	35–40 mph	45–55 mph	> 55 mph
< 5	Not required	Not required	Not required	Not required
5	400	200	120	60
10	240	140	80	40
15	160	100	60	Required
20	120	80	Required	Required
25	100	Required	Required	Required
≥ 26	Required	Required	Required	Required
	Left-turn deceleration lanes are required on rural two-lane highways for the following left-turn volumes: • ≤ 30 mph: 26 veh/hr or more • 35 to 40 mph: 21 veh/hr or more • 45 to 55 mph: 16 veh/hr or more • > 55 mph: 11 veh/hr or more			
Notes: *1. Use linear interpolation of left-turn volumes between 5 and 25 veh/hr.* *2. The directional volume in the through lane includes through vehicles and turning vehicles.*				

Table 15. New Mexico (*46*) criteria for deceleration lanes on rural multilane highways.

Left-Turn Volume[1] (veh/hr)	Left-Turn Deceleration Lane			
	Minimum Directional Volume in Through Lane (veh/hr/ln)[2]			
	≤ 30 mph	35–40 mph	45–55 mph	> 55 mph
< 5	Not required	Not required	Not required	Not required
5	400	200	120	60
10	240	140	80	40
15	160	100	60	Required
20	120	80	Required	Required
25	100	Required	Required	Required
30	130	Required	Required	Required
35	110	Required	Required	Required
≥ 36	Required	Required	Required	Required
	Left-turn deceleration lanes are required on rural multilane highways for the following left-turn volumes: • ≤ 30 mph: 36 veh/hr or more • 35 to 40 mph: 26 veh/hr or more • 45 to 55 mph: 21 veh/hr or more • > 55 mph: 16 veh/hr or more			
Notes: *1. Use linear interpolation of left-turn volumes between 5 and 35 veh/hr.* *2. The directional volume in the through lane includes through vehicles and turning vehicles.*				

The *Iowa Design Manual* material is shown in Table 16. Note the left-turning volume criteria are for a deceleration lane rather than the left-turn bay. Left-turn lanes are to be provided at all Type A entrances (greater than 150 veh/hr) and Type B entrances (moderate traffic volume, 20 to 150 veh/hr).

Oregon and South Dakota have similar criteria. Table 17 and Table 18 show South Dakota's procedure. Both states consider vehicular volume, crash experience, and special cases.

Table 16. Iowa (*47*), left-turn lane material.

Left-turn lanes
Left-turn lanes provide storage in the median for left-turning vehicles, or when warranted, deceleration outside of the through traffic lanes for left-turning vehicles. All Type "A" and high volume Type "B" entrances should have left-turn lanes provided; see Section 3E-2 of this manual. If a left turn deceleration is not warranted, a left turn storage lane should be provided. Normally, left-turn lanes are designed as parallel lanes.

Left Turn Deceleration Lane Warrants
The basic guidelines for when left turn deceleration lanes are warranted involve mainline turning and approach volume, and intersection location.

Turning and approach volume
A left turn deceleration lane may be warranted if left turning traffic flow rate is greater than 30 vehicles per hour measured over a minimum of 15 minutes and either:

a. approach volume is greater than 400 vehicles per hour, or
b. approach truck traffic volume is greater than 40 vehicles per hour.

Intersection location
Intersection location may warrant a left turn deceleration lane even if turning and approach volumes do not. To improve operational efficiency, left turn deceleration lanes should be considered for intersections located within approximately 5 miles (8 kilometers) of an urban area with a population of 20,000 or greater. Other locations where left turn deceleration lanes may be judged to be warranted by the PMT include schools, main entrances for towns, shopping areas, housing developments, attraction locations such as recreational areas, and locations that would have special users such as truck traffic or campers. Special attention should be given to intersections serving locations that attract elderly drivers such as drug stores, grocery stores, retirement developments, medical facilities, nursing homes, etc.

Entrance Types
Type "A" entrance. An entrance developed to carry sporadic or continuous heavy concentrations of traffic. Generally, a Type "A" entrance carries in excess of 150 vehicles per hour. An entrance of this type would normally consist of multiple approach lanes and may incorporate a median. Possible examples include racetracks, large industrial plants, shopping centers, subdivisions, or amusement parks.
Type "B" entrance. An entrance developed to serve moderate traffic volumes. Generally, a Type "B" entrance carries at least 20 vehicles per hour but less than 150 vehicles per hour. An entrance of this type would normally consist of one inbound and one outbound traffic lane. Possible examples include service stations, small businesses, drive-in banks, or light industrial plants.
Type "C" entrance. An entrance developed to serve light traffic volumes. Generally, a Type "C" entrance carries less than 20 vehicles per hour. An entrance of this type would not normally accommodate simultaneous inbound and outbound vehicles. Possible examples include residential, farm or field entrances.

Table 17. South Dakota (*50*) introduction material and criterion 1.

TURN LANE WARRANTS

Left-turn lane Criteria — Unsignalized Intersections

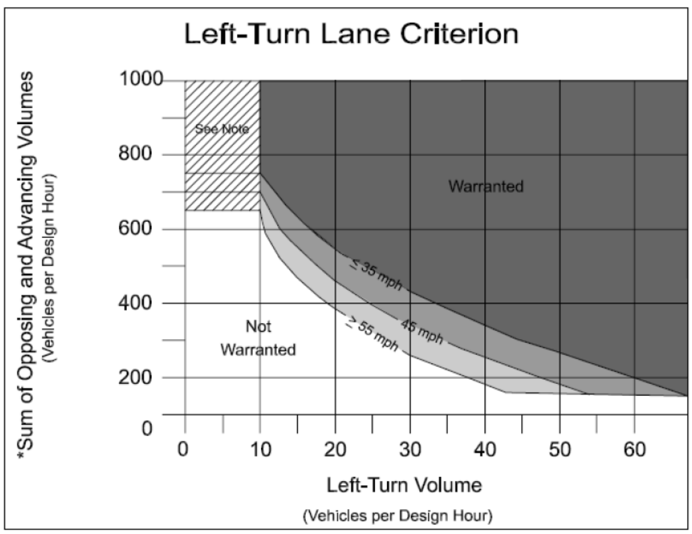
Left-turn lanes should be provided where through and turning volumes create an operational or a potential accident problem.

Left-Turn Lane Evaluation Process

- A left-turn lane should be installed if Criteria 1 (Volume), or 2 (Crash), or 3 (Special Cases) are met, unless a subsequent evaluation eliminates it as an option; and
- The left-turn lane complies with access management spacing standards; and
- The left-turn lane conforms to applicable local, regional and/or state design guidelines.

Criterion 1: Vehicular Volume

The vehicular volume criterion is intended for application where the volume of intersecting traffic is the principal reason for considering installation of a left-turn lane. The volume criteria are determined by the Texas Transportation Institute (TTI) curves in Figure 12-11.



Source: Oregon DOT Analysis Procedures Manual 2008

*(Advancing Vol/ # of Advancing Through Lanes)+
(Opposing Vol/ # of Opposing Through Lanes)

Note: The criterion is not met from zero to ten left turn vehicles per hour, but careful consideration should be given to installing a left-turn lane due to the increased potential for accidents in the through lanes. While the turn volumes are low, the adverse safety and operational impacts may require installation of a left turn. The final determination will be based on a field study.

Figure 12-11 Left-Turn Lane Warrants

Table 18. South Dakota (*50*) criterion 2 and 3 and evaluation guidelines.

Criterion 2: Crash Experience

The crash experience criteria are satisfied when:

1. A history of crashes of the type susceptible to correction by a left-turn lane (such as a vehicle waiting to make a left turn from a through lane was struck from the rear). A separate left-turn lane may be warranted if five or more reported intersection related accidents occur within a 12 month period; and
2. The safety benefits outweigh the associated improvement costs; and
3. The installation of the left-turn lane does not adversely impact the operations of the roadway.

Criterion 3: Special Cases

1. Railroad Crossings: If a railroad is parallel to the roadway and adversely affects left turns, a worst case scenario should be used in determining the storage requirements for the left-turn lane design. The left-turn lane storage length depends on the amount of time the side road is closed, the expected number of vehicle arrivals and the location of the crossing or other obstruction. The analysis should consider all of the variables influencing the design of the left-turn lane, and may allow a design for conditions other than the worst case storage requirements, provided safety is not compromised.
2. Geometric/Safety Concerns: Consider sight distance, alignment, operating speeds, nearby access movements and other safety related concerns.
3. Non-Traversable Median: A left-turn lane should be installed for any break in a non-traversable median where left turns are not prohibited.

Evaluation Guidelines

1. The evaluation should indicate the installation of a left-turn lane will improve the overall safety and/or operation of the intersection and the roadway. If these requirements are not met, the left-turn lane should not be installed or, if already in place, removed from operation
2. Alternatives Considered: List all alternatives that were considered, including alternative locations. Briefly discuss alternatives to the left-turn lane considered to diminish congestion/delays resulting in criteria being met.
3. Access Management: Address access management issues such as the long term access management strategy for the roadway, spacing standards, other accesses that may be located nearby, breaks in barrier/curb, etc.
4. Land Use Concerns: Include how the proposed left-turn lane addresses land use concerns and transportation plans.
5. Plan: Include a design plan layout of the location of the proposed left-turn lane.
6. Operational Requirements: Consider storage length requirements, deceleration distance, desired alignment distance, etc.

Chapter 5: At-Grade Intersections of Minnesota's *Road Design Manual* (*48*) discusses left-turn lane policies. Similar to other design manuals, the document discusses where left-turn lanes should be provided (see Table 19). Criteria are presented for urban highways, multilane highways, and two-lane rural highways. For two-lane rural highways, left-turn lanes should be provided when the access is to a public road, an industrial tract, or a commercial center.

Table 19. Minnesota (*48*) left-turn material.

5-3.01.01 Turn Lane Policy at Urban Intersections Because of the operational and safety benefits associated with right and left-turn lanes, it is Mn/DOT's policy that, in urban areas, they be considered wherever construction is economically feasible taking into account amount of right of way needed, type of terrain, and environmentally or culturally sensitive areas. For new construction/reconstruction projects on divided highways, left-turn and right-turn lanes should be considered at all locations where a paved crossover will be constructed. For preservation projects, left-turn lanes should, if feasible, be provided: 1. At all public road median crossovers. 2. At non-public access locations generating high traffic volumes. 3. At locations where accident records confirm the existence of an excessive hazard. 4. At locations determined by the District Traffic Engineer in consideration of accidents, capacity and traffic volumes. 5. Where a median opening is planned or exists, and its continued existence is justified, a left-turn lane may be added regardless of what the access point serves. **5-4.01 Turn Lanes** As with urban highways, the degree of access control greatly influences the accident rate and efficiency of traffic operation on rural highways. Therefore, designers should try to close any unjustified or potentially dangerous access points. However, if an access point is to remain open, adding a turn lane will enhance the operation and safety. 1. In addition to the policies listed below, left-turn and right-turn lanes should be constructed at locations determined by the District Traffic Engineer in consideration of accidents, capacity, and traffic volumes. 2. Where a median opening is planned, or already exists and its continued existence is justified, a left-turn lane may be added regardless of what the access point serves. 3. Turn lanes should be considered at every public road intersection along a stretch of highway if most intersections on the stretch meet the warrants. If most intersections have turn lanes, motorists will come to expect all intersections to have them. **5-4.01.01 Policy on Multi-Lane Highways** 1. Right-turn and left-turn lanes should be standard features at all public access points. 2. Right-turn and left-turn lanes are also warranted if the access point serves an industrial, commercial, or any substantial trip-generating land use, or if the access point serves more than three residential units. **5-4.01.02 Policy on two-lane Rural Highways** 1. Right-turn lanes should be considered when the projected ADT is over 1500, the design speed is 45 mph or higher, and the following: a. At all public road access points. b. If industrial, commercial, or substantial trip generating land use is to be served, or c. If the access serves more than 10 residential units. 2. Left-turn lanes should be provided when the access is to a public road, an industrial tract or a commercial center. 3. The designer, in conjunction with the District Traffic Engineer may select either a channelized or a painted left-turn lane. The selection will be based on a number of factors including accident history, traffic volume, comparative costs, availability of right of way, environmental impacts, and physical features such as sight distance.

Minnesota's *Mn/DOT Access Management Manual* (*52*) also provides warrants for turn lanes. Turn lanes are to be provided at public street connections and driveways in accordance with the *Mn/DOT Road Design Manual*, Section 5-3, and the guidance provided in the *Access Management Manual*. Guidance provided includes the following:

- Warrant 7: Crash History—at high-volume driveways (> 100 trips per day) and all public street connections that demonstrate a history of crashes of the type suitable to correction by a turn lane or turn-lane treatment (typically three or more correctable crashes in one year), or where adequate trial of other remedies has failed to reduce the crash frequency.

- Warrant 8: Corridor Crash Experience—on highway corridors that demonstrate a history of similar crash types suitable to correction by providing corridor-wide consistency in turn-lane use.
- Warrant 9: Vehicular Volume Warrant—at high-volume driveways (> 100 trips per day) and all public street connections on high-speed highways (posted speed ≥ 45 mph) that satisfy the criteria in Table 20.

Table 20. Minnesota *Mn/DOTAccess Management Manual* (*52*) warrant for left-turn lanes.

Two-Lane Highway ADT	Four-Lane Highway ADT	Cross-Street or Driveway ADT	Turn-Lane Requirement
1500 to 2999	3000 to 5999	> 1500	Left-turn lane warranted
3000 to 3999	6000 to 7999	> 1200	Left-turn lane warranted
4000 to 4999	8000 to 9999	>1000	Left-turn lane warranted
5000 to 6499	10,000 to 12,999	> 800	Left-turn lane warranted
≥ 6500	≥ 13,000	100 to 400	Left-turn or bypass lane warranted
≥ 6500	≥ 13,000	≥ 400	Left-turn lane warranted
Highway AADT one year after opening. Posted speed 45 mph or greater.			

The *Nevada Access Management System and Standards* (*23*) presents the values provided in the *Green Book* for two-lane roads. The document also has criteria for multilane undivided (see Table 21) and divided (see Table 22) roads. The tables list the projected 20-year design-hour volumes (DDHV) of traffic "which necessitate the installation of left turn lanes."

Table 21. Nevada (*23*) left-turn lane requirements for multilane roads (unsignalized).

Opposing Volume (DDHV)	Advancing Volume (DDHV) for Left-Turn Percentages of:			
	5%	10%	20%	30%
800	140	110	80	70
600	220	160	120	100
400	350	250	190	160
200	530	380	290	250
100	650	480	350	310

Table 22. Nevada (*23*) left-turn lane requirements for multilane divided roads (unsignalized).

Opposing Volume (DDHV)	Advancing Volume (DDHV) for Left-Turn Percentages of:			
	5%	10%	20%	30%
800	210	150	110	100
600	340	240	180	150
400	520	380	290	250
200	800	580	440	390
100	1000	720	550	480

Financial Consideration

The Minnesota design manual also uses the term “economically feasible,” which is a concept not discussed in most manuals. The Georgia manual mentions costs in association with left-turn lanes.

Specific Offices within the Department of Transportation

Three states’ manuals state that agencies within the department make the decision. For example, the *Arizona Roadway Design Guidelines* (*53*) states: “Traffic Engineering Group will analyze the traffic movements and other factors at an intersection to determine the need for a separate left-turn lane(s) and establish the vehicle storage requirements for the lane(s).” The *California Highway Design Manual* in Chapter 400: Intersections at Grade (*54*) states: “The District Traffic Branch normally establishes the need for left-turn lanes.” Minnesota (*52*) states: “Left-turn and right-turn lanes should be constructed at locations determined by the District Traffic Engineer in consideration of accidents, capacity, and traffic volumes.”

No Information

During this review, the research team did not find information on left-turn warrants in online documents for Hawaii, Idaho, Kansas, Maine, Maryland, Michigan, New Hampshire, North Carolina, North Dakota, Oklahoma, Rhode Island, South Carolina, Vermont, West Virginia, Wisconsin, or Wyoming. The research team did not find online manuals for Alabama or Arkansas.

RIGHT-TURN LANE INSTALLATION GUIDELINES

Several studies have addressed warrants for the installation of right-turn lanes. The methodology used to develop warrants for right-turn lanes may have application to the development of warrants for left-turn lanes.

Guidelines Based on Volume

NCHRP Report 279 (*10*) provides a summary of the current (mid-1980s) practice in providing exclusive right-turn lane (see Table 23). The report notes:

> No specific warrants or guidelines are apparent for low speed, urban intersections. Engineers generally rely on capacity analyses and accident experience when considering right-turn lanes. In rural areas, focus is primarily on a combination of through and right-turning volume.

Table 23. Summary of state design practice in providing right-turn lanes on rural highways (*10*).

State	Condition Warranting Right-Turn Lane off Major (Through) Highway		
	Through Volume	**Right-Turn Volume**	**Highway Conditions**
Alaska	NA	DHV = 25 veh/hr	Not provided
Idaho	DHV = 200 veh/hr	DHV = 5 veh/hr	2 lane
Michigan	NA	ADT = 600 veh/day	2 lane
Minnesota	ADT = 1500 vpd	All	Design speed > 45 mph
Utah	DHV = 300 veh/hr	ADT = 100 veh/day	2 lane
Virginia	DHV = 500	DHV = 40 mph	2 lane
	All	DHV = 120 veh/hr	Design speed > 45 mph
	DHV = 1200 veh/hr	DHV = 40 veh/hr	4 lane
	All	DHV = 90 veh/hr	4 lane
West Virginia	DHV = 500 veh/hr	DHV = 250 veh/hr	Divided highway
Wisconsin	ADT = 2500 veh/day	Crossroad ADT = 1000 veh/day	2 lane

DHV = design hourly volume
ADT = average daily traffic
NA = not applicable

Guidelines Based on Risk

Yang (*55*) presented a methodology for establishing volume warrant conditions for free right-turn lanes on two-lane roadways based on a risk probability of a potential rear-end crash involving a decelerating right-turn vehicle and the immediately following advancing through volume. Yang developed an equation that can be used to calculate the total approach volume when a free right-run lane on a two-lane roadway is warranted. The equation is a function of:

- Time required to avoid possible rear-end crash:
  - Brake reaction time to surprise event (assumed 1.5 sec),
  - Turning speed of right-turn vehicle (assumed 10 mph),
  - Operating speed of advancing through vehicles, and
  - Uniform deceleration rate (assumed 11.2 ft/sec^2);
- Percent of right turns in total approach volume;
- Minimum headway (assumed 0.66 sec); and
- Risk probability (assumed 0.05).

$$V = \frac{-3600}{\dfrac{t_C - \tau}{\ln\left(1 - \dfrac{P_r}{R(1-R)}\right)} + \tau} \qquad (1)$$

$$t_c = 1.5 + 0.278 \times \frac{s_r - s_t}{a} \qquad (2)$$

Where:

- V = total approach volume (veh);
- t_c = time required to avoid possible rear-end crash (sec);
- s_r = operating speed of advancing through vehicles (km/h [1.0 mph = 1.6 km/h]);
- s_t = turning speed of right-turn vehicles (km/h) (assumed to be 16 km/h [10 mph]);
- a = uniform deceleration rate (m/sec^2) (assumed to be 3.4 m/sec^2 [11.2 ft/sec^2]);
- τ = minimum headway (sec) (assumed to be 0.66 sec);
- P_r = risk probability, i.e., the probability of having a less-than-t_c arrival interval between a right-turning vehicle and its immediately following advancing through vehicle (assumed to be 0.05); and
- R = percent of right turns in the total approach volume.

The limiting probabilities of a through vehicle arriving behind a stopped left-turning vehicle assumed in the Harmelink model are 0.02 for 40 mph, 0.015 for 50 mph, and 0.010 for 60 mph. Yang argues that these values may not be appropriate because a right-turn vehicle does not come to a complete stop and await an acceptable gap in the opposing traffic stream as a left-turn vehicle does and will clear the travel lane in an expectedly short time period. Therefore, he recommends a higher limiting risk probability of 0.05. Using the 0.05 value, he developed a set of warrant curves. Yang's proposed risk probability was evaluated using data available in a previous NCHRP study, and a fair agreement was found between the assumed risk probability and the observed percent of through vehicles impacted by right-turn vehicles. If an agency prefers different assumptions, for example the minimum headway, Yang's equation can be used to determine the warrants for that area.

Guidelines Based on Benefit/Cost

Glennon et al. (*56*) conducted a benefit/cost analysis of right-turn lanes at driveways. The results of the analysis indicated that right-turn lanes are cost-effective at driveways when the driveway volume is at least 1000 veh/day with at least 40 right turns in the driveway during peak periods, the roadway average daily traffic is at least 10,000 veh/day, and the roadway speed is at least 35 mph.

McCoy et al. (*57*) developed guidelines for the use of right-turn lanes at access points on urban two-lane and four-lane roadways. The guidelines define the design-hour traffic volumes for which the benefits of right-turn lanes exceed their costs. The benefits used in the analysis are the operational and accident cost savings that right-turn lanes provide road users. The operational cost savings are those associated with the reductions in stops, delays, and fuel consumption experienced by through traffic as determined through simulation. The guidelines define the right-turn design-hour volume required to justify a right-turn lane as a function of the following factors:

- Directional design-hour volume,
- Roadway speed,
- Number of lanes on the roadway, and
- Right-of-way costs.

Potts et al. (*58*) in 2007 described economic-analysis procedures that identify unsignalized intersections and major driveways at which provision of right-turn lanes is cost-effective. Table 24 shows the equation used in the evaluation.

Table 24. Potts et al. (*58*) equation used to determine benefit/cost ratio.

$$B/C = \frac{\left[\left(DR(V_T, V_{RT}, K)\right) \times n \times DC + AF(V_T, V_{RT}) \times (1 - AMF_{RT}) \times n \times AC\right] \times USPWF(i,n)}{CC} \quad (3)$$

$$USPWF(i,n) = \frac{(1+i)^n - 1}{i(1+i)^n} \quad (4)$$

Where:

B/C	=	benefit-cost ratio;
$DR(V_T, V_{RT}, K)$	=	delay reduction (veh-sec/hr) from right-turn lane installation as a function of V_T, V_{RT}, and K;
V_T	=	peak-hour through traffic volume (veh/hr) on the major road;
V_{RT}	=	peak-hour volume turning right at the location in question (veh/hr);
K	=	design-hour factor, the percentage of ADT during the peak hour;
n	=	service life of right-turn lane (year);
DC	=	user cost savings from delay reduction ($/veh-hr);
$AF(V_T, V_{RT})$	=	expected accident frequency (accidents/year) at location of interest with no right-turn lane present as a function of V_T and V_{RT};
AMF_{RT}	=	accident modification factor for right-turn lane installation;
AC	=	user cost savings from accident reduction ($/accident);
$USPWF(i,n)$	=	uniform series present-worth factor as a function of i and n (see above equation);
i	=	minimum attractive rate of return expressed as a decimal (i.e., 0.04 for a 4 percent return); and
CC	=	estimated construction cost ($) for a right-turn lane.

The operational effects were determined using a computer simulation study of motor vehicles and pedestrians at right-turn lanes. Table 25 lists the delay reductions found for two-lane arterials, and Table 26 lists the delay reductions for four-lane arterials. The estimated reduction in delay to through vehicles provided by a right-turn lane typically ranged from 0 to 6 sec/through veh. The delay reduction is more pronounced as the mainline speeds, through volumes, and right-turn volumes increase.

Pedestrian activity at unsignalized intersections and driveways can have a substantial impact on delay to through vehicles due to right-turning vehicles having to yield to pedestrians. The findings by Potts et al. found delay deductions ranging between 0.4 and 6.0 sec/through veh. Table 27 lists the delay reductions.

The delay values listed in Table 25 to Table 27 are expressed on an hourly basis. To expand these estimates from a single-hour period to an entire year, assumptions were made about the distribution of traffic on typical weekdays and typical weekend days. Table 28 shows an example for a site with a peak-hour through volume of 2200 veh/hr and a peak-hour right-turn volume of 220 veh/hr.

The service life of the right-turn lane was assumed to be 20 years. For the analysis, $9.00 per hour (or 50 percent of the U.S. average wage) was used.

Table 25. Potts et al. (*58*) delay reduction provided by provision of a right-turn lane on a two-lane arterial (sec/through veh).

Speed (mph)	Through Volume (veh/hr)	Right-Turn Volume (veh/hr)						
		0	50	100	150	200	250	300
35	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	200	0.0	0.1	0.2	0.3	0.4	0.5	0.5
	400	0.0	0.2	0.4	0.5	0.7	0.9	1.1
	600	0.0	0.2	0.4	0.6	0.8	1.0	1.1
	800	0.0	0.2	0.4	0.6	0.9	1.1	1.4
	1000	0.0	0.2	0.5	0.8	1.2	1.6	2.1
	1200	0.0	0.3	0.6	1.0	1.5	2.1	1.8
	1400	0.0	0.4	0.9	1.6	2.3	3.2	4.3
45	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	200	0.0	0.2	0.3	0.5	0.6	0.8	0.9
	400	0.0	0.3	0.6	0.9	1.2	1.6	1.9
	600	0.0	0.3	0.6	1.0	1.3	1.6	1.9
	800	0.0	0.4	0.7	1.1	1.5	1.9	2.3
	1000	0.0	0.4	0.9	1.3	1.8	2.3	2.9
	1200	0.0	0.5	1.0	1.6	2.2	2.8	3.5
	1400	0.0	0.6	1.2	1.8	2.5	3.3	4.0
55	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	200	0.0	0.2	0.4	0.7	0.9	1.1	1.3
	400	0.0	0.4	0.9	1.3	1.7	2.2	2.6
	600	0.0	0.5	0.9	1.4	1.9	2.3	2.8
	800	0.0	0.5	1.0	1.6	2.1	2.6	3.1
	1000	0.0	0.6	1.2	1.9	2.5	3.1	3.7
	1200	0.0	0.7	1.4	2.1	2.8	3.6	4.3
	1400	0.0	0.9	1.8	2.8	3.8	4.9	6.1

Table 26. Potts et al. (*58*) delay reduction provided by provision of a right-turn lane on a four-lane arterial (sec/through veh).

Speed (mph)	Through Volume (veh/hr)	Right-Turn Volume (veh/hr)								
		0	50	100	150	200	250	300	350	400
35	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	200	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.1	0.1
	400	0.0	0.0	0.0	0.1	0.1	0.1	0.1	0.1	0.1
	600	0.0	0.0	0.1	0.1	0.1	0.1	0.2	0.2	0.2
	800	0.0	0.0	0.1	0.1	0.1	0.1	0.2	0.2	0.2
	1000	0.0	0.0	0.1	0.1	0.1	0.2	0.2	0.2	0.3
	1200	0.0	0.0	0.1	0.1	0.2	0.2	0.2	0.3	0.3
	1400	0.0	0.0	0.1	0.1	0.2	0.2	0.3	0.3	0.3
	1600	0.0	0.1	0.1	0.1	0.2	0.2	0.3	0.3	0.4
	1800	0.0	0.1	0.1	0.2	0.2	0.3	0.3	0.4	0.4
	2000	0.0	0.1	0.1	0.2	0.2	0.3	0.4	0.4	0.5
	2200	0.0	0.1	0.1	0.2	0.3	0.3	0.4	0.2	0.6
45	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	200	0.0	0.0	0.0	0.0	0.0	0.1	0.1	0.1	0.1
	400	0.0	0.0	0.0	0.1	0.1	0.1	0.1	0.1	0.2
	600	0.0	0.0	0.1	0.1	0.1	0.1	0.2	0.2	0.2
	800	0.0	0.0	0.1	0.1	0.1	0.2	0.2	0.2	0.3
	1000	0.0	0.0	0.1	0.1	0.2	0.2	0.2	0.3	0.3
	1200	0.0	0.0	0.1	0.1	0.2	0.2	0.3	0.3	0.4
	1400	0.0	0.1	0.1	0.2	0.2	0.3	0.3	0.4	0.4
	1600	0.0	0.1	0.1	0.2	0.2	0.3	0.4	0.4	0.5
	1800	0.0	0.1	0.1	0.2	0.3	0.3	0.4	0.5	0.5
	2000	0.0	0.1	0.2	0.2	0.3	0.4	0.4	0.5	0.6
	2200	0.0	0.1	0.2	0.2	0.3	0.4	0.5	0.6	0.6
55	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	200	0.0	0.0	0.0	0.0	0.1	0.1	0.1	0.1	0.1
	400	0.0	0.0	0.0	0.1	0.1	0.1	0.1	0.1	0.2
	600	0.0	0.0	0.1	0.1	0.1	0.1	0.2	0.2	0.2
	800	0.0	0.0	0.1	0.1	0.2	0.2	0.2	0.2	0.3
	1000	0.0	0.0	0.1	0.1	0.2	0.2	0.3	0.3	0.3
	1200	0.0	0.1	0.1	0.2	0.2	0.3	0.3	0.45	0.4
	1400	0.0	0.1	0.1	0.2	0.3	0.3	0.4	0.4	0.5
	1600	0.0	0.1	0.1	0.2	0.3	0.3	0.4	0.5	0.5
	1800	0.0	0.1	0.2	0.2	0.3	0.4	0.5	0.5	0.6
	2000	0.0	0.1	0.2	0.3	0.4	0.4	0.5	0.6	0.7
	2200	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8

Table 27. Potts et al. (*58*) additional delay reduction provided by right-turn lane where pedestrian activity is present (sec/through veh).

Pedestrian Activity Level (ped/hr)	Right-turn Volume (veh/hr)	Through Volume (veh/hr)			
		400	600	800	1200
50	100	0.4	0.5	0.6	0.9
	200	0.9	1.0	1.3	2.1
100	100	0.6	0.6	0.7	1.3
	200	1.2	1.3	1.5	3.1
200	100	0.8	0.9	1.1	2.2
	200	1.7	2.1	2.5	6.0

Table 28. Potts et al. (*58*) example of how annual delay reduction benefit was calculated.

Period	Number of Hours in Weekday (Weekend)	Hourly (Total) Percent of ADT	Traffic Volumes in the Direction of the Right-Turn Lane (veh/hr)		Delay Reduction (sec/ through veh)	Number of Hours per Year	Annual Delay Reduction Benefit (veh-sec/year)
			Through	Right Turn			
AM peak	2 (0)	10.0 (20.0)	2200	220	0.42	522	493,812
PM peak	2 (0)	10.0 (20.0)	2200	220	0.42	522	493,812
Off-Peak	5 (9)	5.8 (29.0)	1276	127	0.13	2241	386,607
Evening	7 (7)	2.6 (18.2)	572	57	0.03	2555	47,817
Night	8 (8)	1.6 (12.8)	352	35	0.01	2920	12,735
Total (24 Hours)	24 (24)	(100)	22,000	2,200		8,760	1,434,783

The annual accident frequency for an unsignalized intersection was estimated from safety performance functions (SPFs) developed for FHWA's Safety Analyst project as stated in its work on right-turn lanes (*58*). The SPFs used in the analysis are listed in Table 29. The safety benefits are based on a study by Harwood et al. (*2*) that developed accident modification factors (AMFs) for the installation of right-turn lanes on major-road approaches to rural and urban intersections (see Table 30). The data used to compute accident cost savings are presented in Table 31.

The cost of constructing a right-turn lane was estimated as $100,000. The authors note that this is a conservative value, which should assure that the analysis results are not overly optimistic.

Table 29. Potts et al. (*58*) equations for predicting accident frequency.

Intersection Type	Equations	
Three-leg intersections with minor-road STOP control	$AF = 0.00475\,(ADT_{major})^{0.34}\,(ADT_{minor})^{0.28}$	(5)
	$ADT_{major} = (2V_T + 2V_{RT})/K$	(6)
	$ADT_{minor} = 2V_{RT}$	(7)
Four-leg intersection with minor-leg STOP control	$AF = 0.0442\,(ADT_{major})^{0.27}\,(ADT_{minor})^{0.16}$	(8)
	$ADT_{major} = (2V_T + 2V_{RT})/K$	(9)
	$ADT_{minor} = 4V_{RT}$	(10)

Where:

- AF = annual accident frequency for unsignalized intersection or driveway,
- ADT_{major} = average daily traffic volume (veh/day) of the major road (i.e., the road on which the right-turn lane is installed,
- ADT_{minor} = average daily traffic volume (veh/day) on the minor leg of the intersection,
- V_T = peak-hour through traffic volume (veh/hr) on the major road,
- V_{RT} = peak-hour volume turning right at location in question (veh/hr), and
- K = design-hour factor, the percentage of ADT during the peak hour.

Assumptions:

- While the SPFs were developed for unsignalized intersections, they are also considered appropriate for application to unsignalized driveways.
- Traffic volumes in both directions of travel on the major road are the same.
- The right-turn volume from the minor road onto the major road is the same as the right-turn volume from the major road onto the minor road that uses the right-turn lane of interest.
- The left-turn volumes entering and leaving the major road are the same as the right-turn volumes.
- At four-leg intersections the turning volumes to and from the minor road are the same on both sides of the major roads.
- At four-leg intersections there is no crossing traffic between the minor-road legs.

Table 30. Potts et al. (*58*) accident modification factors for right-turn lanes.

Intersection Traffic Control	Number of Major-Road Approaches on Which Right-Turn Lanes Are Installed	
	One Approach	Both Approaches
Stop Sign on Minor Road	0.86	0.74
Traffic Signal	0.96	0.92

Table 31. Accident cost and severity distributions (*58*).

Accident Severity	Cost per Accident	Accident Severity Distribution (%)	
		3-Leg Intersection	4-Leg Intersection
Fatal Accident	$3,300,000	0.2	0.4
A Injury Accidents	$228,000	1.4	2.5
B Injury Accidents	$46,000	11.9	11.7
C Injury Accidents	$24,000	21.5	20.5
Property Damage Only (PDO) Accidents	$2,500	65.0	64.9
	Accident cost	$22,000	$31,000

Guidelines Based on a Combination of Approaches

Hai and Thakkar (*59*) developed criteria for right-turn lanes based on speed differential in the outside lane caused by right-turning vehicles. Simulation provided an estimate of the affected through volume in the outside lane and the drop in speed for combinations of outside-lane through movement volume, right-turning volume, and through movement speed. These results were used to derive the critical right-turn volumes as shown in Table 32. Also considered was an assumed relationship between the magnitude of speed deviation and crashes. Critical right-turn volumes were estimated at two levels of benefit-cost ratios. In their study, a benefit-cost ratio of 2.2 was assumed to be associated with a savings of three crashes per year. The 1.5 benefit-cost ratio represents a saving in crashes of 2.2 per year.

Table 32. Hai and Thakkar (*59*) critical right-turn volumes for different operating speeds.

Total Volume (veh/hr)	**Critical Right-Turn Volume (veh/hr)**									
	B/C = 2.2	**B/C = 1.5**	**B/C = 2.2**	**B/C = 1.5**	**B/C = 2.2**	**B/C = 1.5**	**B/C = 2.2**	**B/C = 1.5**	**B/C = 2.2**	**B/C = 1.5**
Speed (mph):	35	35	40	40	45	45	50	50	55	55
400	175	120	150	100	120	70	75	55	60	40
500	150	100	125	80	95	50	65	45	50	30
600	125	80	100	60	75	40	55	35	40	25
700	105	65	80	50	60	35	45	30	35	22
800	90	60	70	45	53	30	40	25	30	20
900	80	50	60	40	45	27	35	23	27	17
1000	70	45	55	35	40	25	30	20	23	15
B/C = benefit/cost ratio										

CHAPTER 3

LITERATURE REVIEW—LEFT-TURN LANE DESIGN

This chapter contains information on the design of left-turn lanes, reviewing information from state design manuals and national reference documents, as well as recent research.

AASHTO *GREEN BOOK* MATERIAL

AASHTO's *Green Book* (*5*) contains general guidance for use in determining the design values for left-turn lanes, also called auxiliary lanes in the book. Many details in the states' design guidelines are based on the *Green Book* text and figures, with some variations. The *Green Book* text on these design guidelines is summarized in the remainder of this section; the original text is supported within the *Green Book* by Exhibits 9-95 to 9-98.

Auxiliary lanes should be at least 10 ft wide and desirably should equal that of the through lanes. Where curbing is to be used adjacent to the auxiliary lane, an appropriate curb offset should be provided.

The length of the auxiliary lanes for turning vehicles consists of three components: entering taper, deceleration length, and storage length. Desirably, the total length of the auxiliary lane should be the sum of the length of these three components. Common practice, however, is to accept a moderate amount of deceleration within the through lanes and to consider the taper length as a part of the deceleration within the through lanes.

Deceleration Length

Provision for deceleration clear of the through traffic lanes is a desirable objective on arterial roads and streets and should be incorporated into design whenever practical. The approximate total lengths needed for a comfortable deceleration to a stop from the full design speed of the highway are as follows: for design speeds of 30, 40, 45, 50, and 55 mph, the desirable deceleration lengths of the auxiliary lane are 170, 275, 340, 410, and 485 ft, respectively, based on grades of less than 3 percent.

On many urban facilities, it is not practical to provide the full length of an auxiliary lane for deceleration, and in many cases the storage length overrides the deceleration length. In such cases, at least part of the deceleration must be accomplished before entering the auxiliary lane. Inclusion of the taper length as part of the deceleration distance for an auxiliary lane assumes that an approaching turning vehicle can decelerate comfortably up to 10 mph in a through lane before entering the auxiliary lane. Shorter auxiliary lane lengths increase the speed differential between turning vehicles and through traffic. A 10-mph differential is commonly considered acceptable on arterial roadways. Higher speed differentials may be acceptable on collector highways and streets due to higher levels of driver tolerance for vehicles leaving or entering the roadway due to slow speeds or high volumes. Therefore, the lengths given above should be accepted as a desirable goal and should be provided where practical. The lengths stated above are applicable to both left- and right-turning lanes, but the approach speed is usually lower in the right lane than in the left lane.

Storage Length

The auxiliary lane should be sufficiently long to store the number of vehicles likely to accumulate during a critical period. The storage length should be sufficient to avoid the possibility of left-turning vehicles stopping in the through lanes waiting for a signal change or for a gap in the opposing traffic flow.

At unsignalized intersections, the storage length, exclusive of taper, may be based on the number of turning vehicles likely to arrive in an average 2-minute period within the peak hour. Space for at least two passenger cars should be provided; with over 10 percent truck traffic, provisions should be made for at least one car and one truck. The 2-minute waiting time may need to be changed to some other interval that depends largely on the opportunities for completing the left-turn maneuver. These intervals, in turn, depend on the volume of opposing traffic. Where the volume of turning traffic is high, a traffic signal is often needed.

Taper

On high-speed highways it is common practice to use a taper rate that is between 8:1 and 15:1 (longitudinal:transverse [L:T]). Long tapers approximate the path drivers follow when entering an auxiliary lane from a high-speed through lane. However, long tapers tend to entice some through drivers into the deceleration lane—especially when the taper is on a horizontal curve. Long tapers constrain the lateral movement of a driver desiring to enter the auxiliary lanes. This problem primarily occurs on urban curbed roadways.

For urbanized areas, short tapers appear to produce better “targets” for the approaching drivers and to give more positive identification to an added auxiliary lane. Short tapers are preferred for deceleration lanes at urban intersections because of slow speeds during peak periods. The total length of taper and the deceleration length should be the same as if a longer taper was used. This results in a longer length of full-width pavement for the auxiliary lane. This type of design may reduce the likelihood that entry into the auxiliary lane will spill back into the through lane. Recent practice has trended toward using a standard, typically short, taper length instead of a taper ratio. Municipalities and urban counties are increasingly adopting the use of taper lengths such as 100 ft for a single turn lane and 150 ft for a dual turn lane for urban streets.

Straight-line tapers are frequently used. The taper rate may be 8:1 (L:T) for design speeds up to 30 mph and 15:1 (L:T) for design speeds of 50 mph. Straight-line tapers are particularly applicable where a paved shoulder is striped to delineate the auxiliary lane. Short, straight-line tapers should not be used on curbed urban streets because of the probability of vehicles hitting the leading end of the taper with the resulting potential for a driver losing control. A short curve is desirable at either end of long tapers but may be omitted for ease of construction. Where curves are used at the ends, the tangent section should be about one-third to one-half of the total length.

Width

Median widths of 20 ft or more are desirable at intersections with single median lanes, but widths of 16 to 18 ft permit reasonably adequate arrangements. Where two median lanes are used, a median width of at least 28 ft is desirable to permit the installation of two 12-ft lanes and a 4-ft separator. Although not equal in width to a normal travel lane, a 10-ft lane with a 2-ft curbed separator or with traffic buttons or paint lines, or both, separating the median lane from the opposing through lane may be acceptable where speeds are low and the intersection is controlled by traffic signals.

Offset Left-Turn Lanes

For medians wider than about 18 ft, it is desirable to offset the left-turn lane so that it reduces the width of the divider to 6 to 8 ft immediately before the intersection, rather than to align it exactly parallel with and adjacent to the through lane. This alignment places the vehicle waiting to make the turn as far to the left as practical, maximizing the offset between the opposing left-turn lanes and thus providing improved visibility of opposing through traffic. The advantages of offsetting the left-turn lanes are better visibility of opposing through traffic; decreased possibility of conflict between opposing left-turn movements within the intersection; and more left-turn vehicles served in a given period of time, particularly at a signalized intersection. Parallel offset left-turn lanes may be used at both signalized and unsignalized intersections.

An offset between opposing left-turn vehicles can also be achieved with a left-turn lane that diverges from the through lanes and crosses the median at a slight angle. Tapered offset left-turn lanes provide the same advantages as parallel offset left-turn lanes in reducing sight distance obstructions and potential conflicts between opposing left-turn vehicles and in increasing the efficiency of signal operations. Tapered offset left-turn lanes are normally constructed with a 4-ft nose between the left-turn lane and the opposing through lanes. Tapered offset left-turn lanes have been used primarily at signalized intersections.

Parallel and tapered offset left-turn lanes should be separated from the adjacent through traffic lanes by painted or raised channelization.

STATE MANUALS

This section contains an overview of information from the design manuals, standards, and guidelines governing the design of left-turn lanes in individual states. The information was obtained from the online manuals available at the respective websites of the states' departments of transportation. Not all 50 states are shown in this section; some states did not provide access to the relevant documents through their websites, while others did not have design details in the material contained in the available manuals.

Table 33 provides a summary of the design guidelines and standards referenced in this document. The actual text and figures from those manuals are provided in Appendix B.

Table 33. Summary of left-turn lane design guidelines.

	Queue Storage Length	Entering Taper Length	Deceleration Length	Width	Offset	Dual Turn Lanes	Two-Way Left-Turn Lane	Pedestrians	Indirect Turn Designs	Bypass Lane
Green Book	✓	✓	✓	✓			✓			
Alaska	✓	✓	✓							
Arizona	✓	✓		✓		✓	✓	✓		
California	✓	✓	✓	✓		✓	✓			
Colorado	✓	✓	✓							
Connecticut	✓	✓		✓						✓
Delaware	✓	✓	✓	✓				✓		
Florida				✓				✓		
Georgia	✓	✓	✓	✓		✓				✓
Illinois	✓	✓	✓	✓	✓				✓	
Indiana	✓	✓	✓	✓	✓					✓
Iowa	✓		✓	✓						
Kentucky	✓	✓	✓	✓						
Louisiana	✓	✓								
Maine	✓	✓	✓	✓						
Minnesota	✓	✓	✓	✓						✓
Nebraska	✓	✓	✓	✓	✓					
New Jersey	✓	✓		✓			✓			
Ohio	✓	✓	✓	✓						
Pennsylvania	✓	✓	✓	✓						
Rhode Island				✓						
Tennessee	✓	✓	✓	✓						
Texas	✓	✓	✓	✓			✓	✓		
Wisconsin	✓	✓	✓	✓				✓		

Queue Storage Length

Most states in this review call for a minimum storage length of 50 ft (two passenger cars at 25 ft each), though some have a minimum of 150 ft, which accounts for some taper and deceleration length as well as storage. For additional length greater than the minimum, the state guidelines generally call for sufficient length to store the passenger vehicles expected in a 2-minute period during the peak hour. Adjustments can be made for locations with higher proportions (generally greater than 10 percent) of heavy vehicles. Additionally, a frequently mentioned guideline is that the storage length should provide sufficient space so that neither turning nor through traffic blocks the other.

Entering Taper Length

Two distinct tapers are defined in the state manuals: approach taper length and bay taper length. The approach taper length is commonly defined as V × W, where V = design speed (mph) and W = turn lane width. Some states subdivide this definition into high speed (≥ 45 mph, V × W) and low speed (≤ 40 mph, $W \times V^2 / 60$) conditions. The bay taper length is generally provided as a minimum length between 60 and 120 ft (as short as 50 ft and as long as 200 ft) or a ratio between 8:1 and 15:1; one state defined bay taper length as W × V / 3.

Deceleration Length

The length of the deceleration portion of a left-turn lane is based on the design speed, speed limit, or operating speed of the facility. In the vast majority of cases, the state manuals either implicitly or explicitly use values from the 2004 *Green Book*: for design speeds of 30, 40, 45, 50, and 55 mph, the desirable deceleration lengths of the auxiliary lane are 170, 275, 340, 410, and 485 ft, respectively, based on grades of less than 3 percent. Depending on the publication date of the manual, some states use the values from the 2001 *Green Book*: 230, 330, 430, 550, and 680 ft, respectively.

Width

Specified turn-lane widths are generally determined relative to the functional classification, urban or rural location, and project scope of work. Widths commonly vary from 10 to 12 ft, with a desirable width of 12 ft. On rural and urban high-speed highways, 11 ft is usually the minimum, while 10 ft is permitted only on urban low-speed roads. In many cases, a shoulder of 1.5 to 2.0 ft is also specified, particularly if the lane is adjacent to a curb.

Offset

The states that mention offset characteristics all refer to the benefits of improving turning drivers' line of sight and call for use of offset when wide medians are available. In Nebraska (*35*), the left-turn lanes in 16-ft raised medians should be designed with a 1-ft offset. Wide striping on the right side of the left-turn lane should be used to encourage traffic to move closer to the median.

In Illinois (*30*), offset left-turn lanes can consist of either a tapered design or a parallel design. Figures provided in their manual illustrate the various designs for offset left-turn lanes. In addition, the designer should consider the following:

1. Tapered offset left-turn lanes. The advantages of the tapered offset design versus a parallel lane design without an offset are that the offset design provides better visibility for the turning motorist to the opposing traffic, decreases the possible conflict between opposing left-turning vehicles, and serves more left-turning vehicles in a given time period. In addition, the designer should consider the following:
    a. Guidelines. Provide a tapered offset left-turn lane design where at least two of the following are applicable:

        - The median width is equal to or greater than 40 ft, and only one left-turn lane in each direction on the mainline highway is required for capacity.
        - The current mainline ADT is 1500 or greater, and the left-turn DHV in each direction from the mainline is greater than 60 veh/hr. Under these conditions, vehicles waiting in opposing left-turn lanes have the probability of obstructing each other's line of sight.
        - The intersection will be signalized.
    b. Median widths. Median widths of 40 to 70 ft are allowed to remain in place on existing expressways or multilane facilities. On new construction or reconstruction projects, use a median width of 50 ft and median slopes of 1V:6H.
    c. Curb and gutter. Use an M-4 curb and gutter on all corner and channelizing island, unless signals are placed within the island. In this situation, use an M-6 curb and gutter.
2. Parallel offset left-turn lanes. Parallel offset left-turn lanes offer the same advantages as the tapered design. However, they may be used at intersections with medians less than 40 ft but greater than 13 ft.

In Indiana (*22*), on a four-lane facility with a wide median, slotted left-turn lanes are desirable where the median width is equal to or greater than 24 ft. The designer should consider the following:

1. Slot length. The slotted section of the turn lane should be at least 50 ft long with a minimum of 100 ft. The slotted section should not include the required deceleration distance for the turn lane.
2. Nose width. The nose of the slotted lane should be a minimum of 4 ft plus shoulder- or curb-offset width (or return taper) from the opposing through lanes. The nose position should be checked for interference with the turning paths from the cross street.
3. Slot angle. The angle of the slot should not diverge more than 10 degrees from the through mainline alignment.
4. Island. To delineate the slotted portion, the channelized island for the slotted lane should be a raised corrugated island. Raised pavement markers may be used for further delineation.

Dual Turn Lanes

Dual left-turn lanes are often needed to satisfy high-volume demands. Capacity analysis should be used to identify the need for dual left-turn lanes. Dual left-turn lanes are typically considered at signalized intersections when the peak-hour left-turn volume is 300 vehicles or greater. The decision to use dual left-turn lanes should consider the off-peak periods as well as the peak periods. The off-peak periods may be adversely affected since the use of dual left-turn lanes typically precludes permissive left turns.

Georgia guidelines state that if dual left-turn lanes are included in the design, the following design guidelines should be considered:

- Because of off tracking and the added difficulty involving two-abreast turns, a minimum 30-ft throat width should be provided through the intersection.
- Pavement markings should be provided to guide the path of the turning vehicles.

- The design should be checked to ensure that conflicts are minimized between opposing left-turn maneuvers.

When dual left-turn lanes are located opposite from an approach that does not have a dual left-turn lane, the design should minimize the lateral offset for vehicles traveling straight through the intersection. This can be accomplished by providing a median or striped-out area opposite the dual left-turn lane.

TWLTL

Continuous two-way left-turn lanes (TWLTLs) are often used in urban and fringe urban areas to treat the special capacity and safety concerns associated with left-turn demands at high-density strip developments. Two-way left-turn lanes may be used with either two-lane or multilane highways. The lane width should be no less than that of the through traffic lanes.

In California, the TWLTL is devised to address the special capacity and safety problems associated with high-density strip development. The minimum width is 12 ft; the preferred width is 14 ft. Wider TWLTLs are occasionally provided to conform to local agency standards. However, TWLTLs wider than 14 ft are not recommended, and in no case should the width of a TWLTL exceed 16 ft. Additional width may encourage drivers in opposite directions to use the TWLTL simultaneously.

In New Jersey, lane widths for continuous two-way left-turn median lanes range from 12 ft to 16 ft. The wider pavement width should be used only when raised islands are provided at major intersections with high left-turn demands. A median lane width of 12 ft is desirable where raised islands are not provided at major intersections.

In Texas, TWLTL facilities for suburban roadways should minimally be 14 ft and desirably 16 ft in width. The desirable value of 16 ft width should be used on new location projects or on reconstruction projects where widening necessitates the removal of exterior curbs. The "minimum" value of a 14-ft width is appropriate for restrictive right-of-way projects and improvement projects where attaining "desirable" median lane width would necessitate removing and replacing exterior curbing to gain only a small amount of roadway width. Criteria for the potential use of a continuous TWLTL on a suburban roadway are as follows:

- Future ADT volume of 3,000 vehicles per day for an existing two-lane suburban roadway, 6000 vehicles per day for an existing four-lane suburban roadway, or 10,000 vehicles per day for an existing six-lane suburban roadway; and
- Side road plus driveway density of 10 or more entrances per mile (six or more per kilometer).

When both conditions are met, the use of a TWLTL should be considered. For ADT volumes greater than 20,000 vehicles per day, or where development is occurring and volumes are increasing and are anticipated to reach this level, a raised median design should be considered.

Pedestrians

Several states mention pedestrian considerations in the design of left-turn lanes. Arizona's guidelines state that when left-turn lanes are placed in raised (curbed) medians, a minimum of 4 ft should remain at the nose for pedestrian refuge and placement of traffic control devices.

In Delaware, in urban areas where speeds are low and the intersection is controlled by traffic signals, a 10-ft lane with a 2-ft curbed separator or paint lines, or both, may be acceptable to separate the median lane from the opposing through lane. Where pedestrian use is anticipated, a 6-ft separator should be provided.

Deceleration lanes for left turns in Florida should be provided on all high-speed facilities. These turn lanes should not be excessive or continuous since they complicate pedestrian crossings and bicycle/motor vehicle movements. Storage (or deceleration lanes) to protect turning vehicles should be provided, particularly where turning volumes are significant.

For median left-turn lanes at Texas intersections, a median width of 16 ft (a 12-ft lane plus a 4-ft divider) is recommended to accommodate a single left-turn lane. For maintenance considerations in preventing recurring damage to the divider, the divider should be at least 2 ft. If pedestrians are expected to cross the divider, then the divider should be a minimum of 5 ft wide in order to accommodate a cut-through landing or refuge area that is at least 5 ft by 5 ft. Seven-lane cross sections should be evaluated for pedestrian crossing capabilities.

Wisconsin guidelines state that, for pedestrian accommodation or protection in the median, designers should line up the face of the median nose with the cross-street sidewalk extended.

Indirect Turn Designs

Indiana mentions the option of indirect left turns in its manual. It states that where operational or safety concerns preclude the use of typical left-turn lanes, the designer may consider the use of indirect left turns or jughandles that cross the mainline or intersect the crossroad at a different location. Because these require special consideration and treatment, they must be developed in consultation with the Bureau of Design and Environment.

Bypass Lanes (or Blister Lanes)

Four states (Connecticut, Georgia, Indiana, and Minnesota) discussed bypass lanes, also called blister lanes. The illustrations for the typical design for a bypass area are included in Appendix B.

PREVIOUS RESEARCH

This section contains information from other reference documents and recent research that are national in scope or that have design guidance in addition to that found in state design manuals.

Left-Turn Lane Length

Kikuchi et al. (*60*) developed a proposed procedure for determining the appropriate length of dual left-turn lanes (DLTL) in a 2004 study. The procedure surveyed how drivers chose a lane of the DLTL and analyzed the relationship between lane use and the volume of left-turning vehicles. The procedure formulated the probability that all left-turning vehicles would be able to enter the left-turn lanes and derived the adequate lane length such that the probability of vehicles entering the DLTL is greater than a threshold value. Recommended lengths were presented as a function of left-turn and through volumes for practical application, with the purpose of avoiding left-turn lane overflow and blockage of lane entrance. The results were presented as a range of values in terms of number of vehicles and actual length, which were substantially longer than other methods that proposed a basic multiplier of the length of a comparable single left-turn lane. While this method is proposed for signalized intersections, there may be limited application for accommodations in selected locations with short durations of high left-turn demand (e.g., entrances to special event venues that are unsignalized).

Gard (*61*) conducted a study to develop a set of empirical equations to accurately predict maximum queue lengths at unsignalized intersections. Using traffic data from a set of 15 intersections in California, Gard developed a series of regression equations for the turning movements at an unsignalized intersection. His equations for major-street left turns are shown in Table 34.

Table 34. Gard (*61*) regression equations for major-street left-turn queue length at unsignalized intersections.

Movement	Condition	Equation	
Major-Street Left Turn	Approach volume ≤ 100 veh/hr/PHF	Max. queue = −2.042 + 1.167 ln (AV) + 0.975 × TS	(11)
	Approach Volume > 100 veh/hr/PHF	Max. queue = 4.252 − 1.23 × L + 0.07996 × S + 1.412 × TS – 374.028 / AV + 0.00001144 × AV × CV	(12)

Where:

- AV = approach volume, hourly traffic volume divided by peak hour factor (PHF) for subject movement;
- CV = conflicting volume, hourly traffic volume divided by PHF that conflicts with the subject movement;
- TS = traffic signal presence, a dummy variable with a value of 1 if a traffic signal is located on the major street within 0.25 miles of the subject intersection and a value of 0 otherwise;
- L = lanes, the number of through lanes occupied by conflicting traffic; and
- S = speed, the posted speed limit on the major street (mph).

Gard compared these equations to the procedures found in the 2000 *Highway Capacity Manual*, the monograph in ITE’s 1988 *Transportation and Land Development*, and the 2-minute arrival methodology in the 2001 AASHTO *Green Book*. He found that, of the 70 data points for major-street left turns, his method correctly predicted 34 percent of the observations, and 84 percent were predicted within one vehicle. In contrast, the *Green Book* method predicted 35 percent correctly and 71 percent within one vehicle; the other methods tended to underestimate queues, providing shorter lengths than needed to accommodate queuing traffic.

In a comparison of methods similar to Gard's, Lertworawanich and Elefteriadou (*62*) also developed a method to estimate storage lengths and compared it to the 2001 *Green Book*. The authors' model, based on a Poisson arrival process, considered service times of vehicles arriving at an empty left-turn lane and of vehicles arriving at an occupied left-turn lane, but did not consider the effects of heavy vehicles. They created a series of tables of recommended storage lengths based on a threshold of the probability of overflow, and they compared their results to the *Green Book*. The authors found that, in comparison to their Poisson model, the *Green Book* tended to overestimate the necessary storage lengths until the volumes approached capacity, while both the *Green Book* and Poisson methods underestimated the queue lengths.

NCHRP Report 457 (*12*) developed suggested storage length values using a procedure that was similar to Harmelink's work regarding storage length of left-turn bays at unsignalized intersections. The storage length equation is a function of movement capacity, which is dependent upon assumed critical gap and follow-up gap. *NCHRP Report 457* used a smaller critical gap (4.1 sec as recommended in the *Highway Capacity Manual* compared to the 5.0 or 6.0 sec used by Harmelink for two-lane and four-lane highways, respectively), which resulted in shorter values than those generated by Harmelink.

The Texas *Urban Intersection Design Guide* (*63*) states that the length of left-turn lanes depends on three elements:

- Deceleration length,
- Storage length, and
- Entering taper.

If insufficient room is available for each of these elements, allowing a moderate amount of deceleration length to be included in the taper section is acceptable. Deceleration length assumes that moderate deceleration will occur in the through traffic lane and the vehicle entering the left-turn lane will clear the through traffic lane at a speed of 10 mph slower than through traffic. Where providing this deceleration length is impractical, it may be acceptable to allow turning vehicles to decelerate more than 10 mph before clearing the through traffic lane.

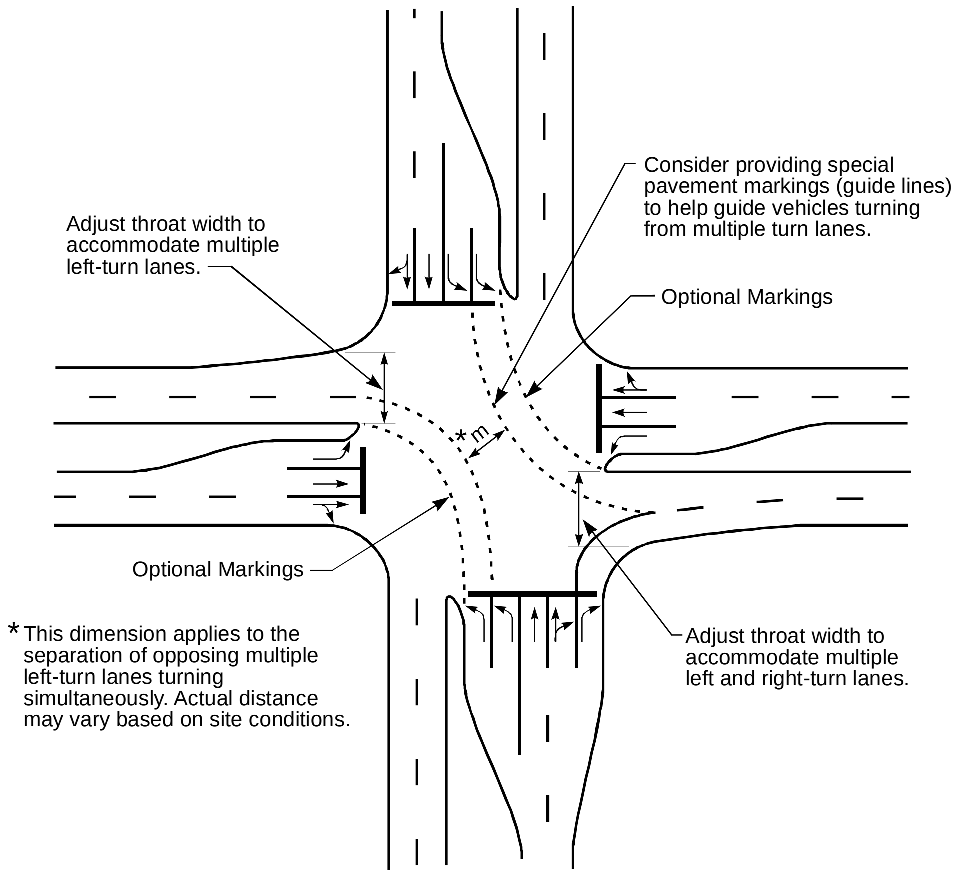
Taper

On high-speed highways it is common practice to use a taper rate that is between 8:1 and 15:1 (L:T) (*5*). Long tapers approximate the path drivers follow when entering a left-turn lane from a high-speed through lane. However, long tapers tend to entice some through drivers into the deceleration lane—especially when the taper is on a horizontal curve. Long tapers also constrain the lateral movement of a driver desiring to enter the turn lanes.

For urban areas, short tapers appear to produce better "targets" for the approaching drivers and to give more positive identification of an added left-turn lane. Short tapers are preferred for deceleration lanes at urban intersections because of slow speeds during peak periods. The total length of taper and the deceleration length should be the same as if a longer taper was used. This results in a longer length of full-width pavement for the auxiliary lane. This type of design may reduce the likelihood that entry into the left-turn lane may spill back into the through lane.

Municipalities and urban counties are increasingly adopting the use of taper lengths such as 100 ft for a single turn lane and 150 ft for a dual turn lane for urban streets (*5*).

If dual left-turn lanes are used, the length required for storage is approximately half that required for single left-turn lanes (*5*). Flexibility in signalization is provided if the left-turn movements are separated as shown in Figure 10 (dimension m, as described in the note) (*63*, *64*). This separation, if sufficient, can allow concurrent dual left-turn phases. Separate dual left-turn phases eliminate the potential problem of overlapping vehicle paths in the intersection.



Source: Fitzpatrick, K., et al., *Urban Intersection Design Guide*, FHWA/TX-05/04365-P2. Copyright Texas Transportation Institute, The Texas A&M University System, College Station, Texas, 2005. Reproduced with permission of the author.

Figure 10. Dual left-turn lane (*63, 64*).

Left-Turn Lane Width

The width of auxiliary lanes should preferably match the width of the through lanes although they should be at least 10 ft wide (*5*). If curbs are present, a curb offset of 1 to 2 ft from the edge of the travel lane to the face of the curb should be used.

To accommodate a single left-turn lane, a median width of 18 ft—a 12-ft lane width plus a 6-ft divider—is recommended. The 6-ft divider may provide a refuge for pedestrians, depending on its design; however, it is not sufficient to fully offset the turn lane (discussed below). If dual left-turn lanes are used, the median opening and crossroad should be sufficiently wide to accommodate both incoming lanes; a median width of 28 to 30 ft—11- to 12-ft lanes plus a 6-ft divider—is recommended (*63*).

Offset Left-Turn Lanes

Vehicles in opposing left-turn lanes can limit each other's views of approaching traffic. The restriction on the sight distance is dependent on the amount and direction of the offset between the opposing left-turn lanes. The offset is measured between the left edge of a left-turn lane and the right edge of the opposing left-turn lane as shown in Figure 11 (*63*).

Benefits of positive offset left-turn lanes include:

- Better visibility of opposing through traffic,
- Improved unprotected left-turn phase,
- Decreased possibility of conflict between opposing left-turn movements within the intersection, and
- Service for more left-turn vehicles in a given period of time (particularly at signalized intersections).

The impact on pedestrian crossings of all roadways should be considered in the design of offset left-turn lanes.

Greater right-of-way width is required to offset left-turn lanes, but research has shown that they can provide significantly greater sight distance for left-turn maneuvers, a particularly critical maneuver for older drivers (*65*). Guidelines were developed for offsetting opposing left-turn lanes at 90-degree intersections on level, tangent sections of divided roadways with 12-ft lanes (see Table 35) (*66*). The minimum offsets in the table are those required to provide opposing left-turning vehicles with adequate sight distances. They are applicable to left-turning passenger cars opposed by either another passenger car or a truck. The desirable offsets are those that provide opposing left-turning vehicles with unrestricted sight distances, and therefore, they are independent of design speed. The guidelines include minimum and desirable offsets when both vehicles are unpositioned, and the left-turning vehicle is unpositioned and the opposing left-turning vehicle is positioned. Positioned vehicles enter the intersection to obtain a better view of oncoming traffic, while unpositioned vehicles remain behind the stop line while waiting to turn left. A previous study found that 60 percent of older drivers did not position their vehicle. Therefore, in areas with high percentages of older drivers, the guidelines based on both vehicles being unpositioned should be used. Likewise, in areas where there are high percentages of trucks, the guidelines based on the opposing left-turning vehicle being a truck should be used.

The guidelines presented in Table 35 typically involve reconstructing the left-turn lanes. Increasing the width of the lane line between the left-turn lane and the adjacent through lanes can also improve the sight distance by encouraging the driver to position the vehicle closer to the median. McCoy et al. (*67*) developed a methodology for determining the width of the left-turn lane line.

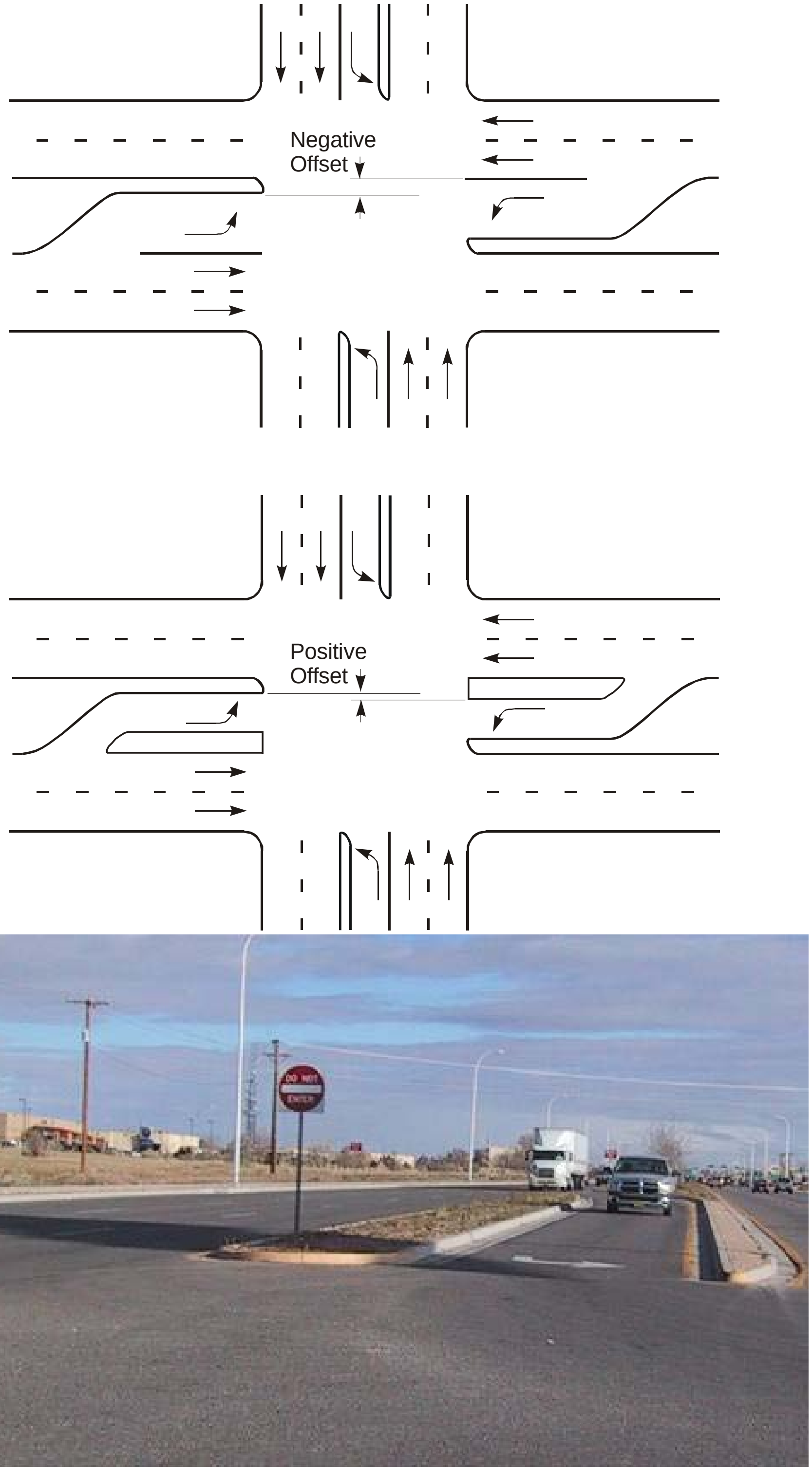


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Figure 11. Examples of offset left-turn lanes (*63*).

Table 35. Tarawneh and McCoy (*66*) guidelines for offsetting opposing left-turn lanes.

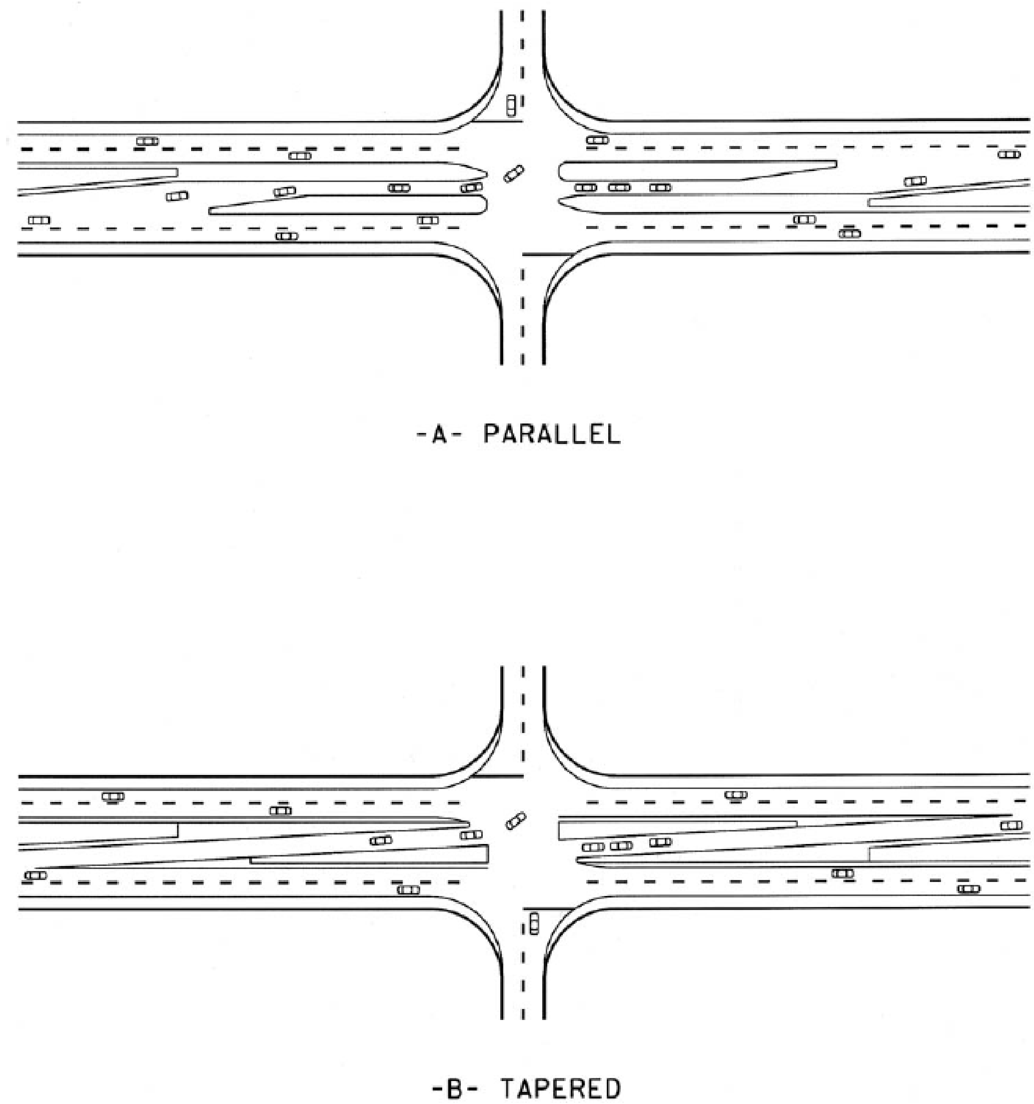
Metric									
Opposing Left-Turn Vehicle		**Minimum Offset (m)**							**Desirable Offset (m)**
		Design Speed (km/h)							
Type	**Location**	50	60	70	80	90	100	110	
Passenger Car	Unpositioned	1.0	1.0	1.1	1.1	1.1	1.2	1.2	1.3
	Positioned	0.2	0.3	0.3	0.4	0.4	0.4	0.4	0.6
Truck	Unpositioned	1.5	1.5	1.5	1.6	1.6	1.6	1.6	1.7
	Positioned	0.8	0.8	0.9	0.9	0.9	1.0	1.0	1.1
U.S. Customary									
Opposing Left-Turn Vehicle		**Minimum Offset (ft)**							**Desirable Offset (ft)**
		Design Speed (mph)							
Type	**Location**	31	37	43	50	56	62	68	
Passenger Car	Unpositioned	3.3	3.3	3.6	3.6	3.6	3.9	3.9	4.3
	Positioned	0.7	1.0	1.0	1.3	1.3	1.3	1.3	2.0
Truck	Unpositioned	4.9	4.9	4.9	5.2	5.2	5.2	5.2	5.6
	Positioned	2.6	2.6	2.9	2.9	2.9	3.3	3.3	3.6

Two types of offset left-turn lanes are typically used: parallel and tapered. Parallel lanes may be used at both signalized and unsignalized intersections, while tapered lanes are usually used only at signalized intersections. An illustration of both types is provided in Figure 12.

Tapered offset left-turn lanes are normally constructed with a 4-ft nose between the left-turn and the opposing through lanes. This median nose can be offset from the opposing through traffic by 2 ft or more with a gradual taper, making it less vulnerable to contact by the through traffic (see Figure 12[b]).

This type of offset is especially effective for the turning radius allowance where trucks with long rear overhangs, such as logging trucks, are turning from the mainline roadway. This same type of offset geometry may also be used for trucks turning right with long rear overhangs (*5*).

Parallel and tapered offset left-turn lanes should be separated from the adjacent through traffic lanes by painted or raised channelization. Adequate advance signing is essential so that drivers recognize the need to enter the turn lane well in advance of the intersection.



Source: *A Policy on Geometric Design of Highways and Streets.* Copyright American Association of State Highway and Transportation Officials, Washington, D.C., 2004. Reproduced with permission of AASHTO.

Figure 12. Parallel and tapered offset left-turn lane (*5*).

Results of a 1996 study by Tarawneh and McCoy (*66*) indicated that driver performance can be adversely affected by offsets that are much less (i.e., more negative) than −2.95 ft. Such large negative offsets significantly increase the size of the critical gaps of drivers turning left and also seem to increase the likelihood of conflicts between left turns and opposing through traffic. Large negative offsets may be particularly troublesome for older drivers and women drivers, who are less likely to position their vehicles within the intersection to see beyond vehicles in the opposing left-turn lane.

The same 1996 study had a somewhat counterintuitive finding. Driver perceptions of the level of comfort were not found to improve with greatly increased offsets. An offset of 5.9 ft was associated with a lower level of comfort and a higher degree of difficulty perceived by drivers than an offset of −2.95 ft, even though the latter provides less sight distance. The study's authors speculated that this reaction might be because the −2.95-ft offset is more common than the 5.9-ft offset.

Intersection Sight Distance

Yan and Radwan (*68*) developed sight distance geometric models for unprotected left-turning vehicles at parallel and taper left-turn lanes. For parallel left-turn lanes, they calculated available sight distance as follows:

$$SD = V_f + D + \frac{\left(\frac{L_t}{2}+m-n-g-V_w\right)\times(V_f+D)}{2n+2g+e+V_w-m} \qquad (13)$$

Where:

- SD = available sight distance (ft);
- V_f = distance from the eye of the driver to the front of the vehicle (ft);
- D = distance between stop bars of opposing left lanes, which is composed of the width of pedestrian corridors and the width of the minor road (ft);
- L_t = width of the opposing through lane (ft);
- m = width of the median (ft);
- n = width of the median nose (ft);
- g = distance from the left side of the left-turn vehicle to the left lane line (ft),
- V_w = width of the opposing left-turn vehicle (ft); and
- e = distance from the eye of the driver to the left side of the vehicle (ft).

The authors produced a table that calculates available sight distance for common dimensions of parallel opposing left-turn lanes, reproduced here as Table 36. They also produced similar equations and tables for parallel lanes with offset and for tapered left-turn lanes.

Table 36. Yan and Radwan (*68*) calculated available sight distance for traditional parallel opposing left-turn lanes.

m (ft)	n (ft)	Lane Width (ft)	D (ft)	Available Sight Distance (ft)
12	0	12	83	1729
13	1	12	83	637
14	2	12	83	419
15	3	12	83	325
16	4	12	83	273
17	5	12	83	240
18	6	12	83	217
19	7	12	83	200
20	8	12	83	187

The models and related analyses focused only on the unprotected phases of a signalized intersection, but the principles are comparable for sight visibility at unsignalized intersections. The authors caution that the absence of stop bars at unsignalized intersections discouraged a direct application of these models because left-turning vehicles' positions could be more flexible before crossing the opposing through traffic.

Alternate Intersection Designs

A number of alternate designs have been proposed and implemented to change the configuration of intersections to improve the efficiency and/or safety of turning movements. One such design is the crossover displaced left-turn (XDL) intersection, also called the continuous flow intersection (CFI). The fundamental design principle of the XDL intersection involves displacement of the left-turn lane to the other side of the opposing through lanes several hundred feet upstream of the intersection. The displaced left-turn lanes are aligned parallel to the through lanes at the intersection. This design results in the simultaneous movement of left-turning traffic with through traffic at the intersection. The key tradeoffs are the need for additional right-of-way to accommodate the displaced lanes and the creation of several smaller ancillary intersections around the primary intersection, which must also be maintained with signing and marking. This design is primarily intended for signalized intersections as an alternative to grade separation, but there may be possible applications for unsignalized intersections at high-speed locations. Jagannathan and Bared (*69*) modeled the performance of three sample XDL intersections in comparison to conventional intersections and found that average intersection delay, average number of stops, average queue length, and capacity all improved with the XDL.

Roundabouts are growing in popularity in the United States, after having been developed in the 1960s in the United Kingdom and used at numerous intersections in other countries, primarily throughout other parts of Europe as well as Australia. Two key characteristics of the modern roundabout include a requirement for entering traffic to yield to circulating traffic and geometric constraints that slow entering vehicles. One result is that traditional left turns are eliminated as all intersection traffic travels around the circulatory roadway in the same direction. A recent NCHRP project (*70*) examined the safety and operation of roundabouts in the United States, with the purpose of producing a set of operational, safety, and design tools, calibrated to U.S. roundabout field data. The researchers found that, with the exception of conversions from all-way-stop-controlled intersections, where crash experience remains statistically unchanged, roundabouts have improved both overall crash rates and, particularly, injury crash rates in a wide range of settings (urban, suburban, and rural) and previous forms of traffic control (two-way stop and signal). Statistical analysis revealed a 35 percent reduction in crashes for all sites studied. Overall, single-lane roundabouts have better safety performance than multilane roundabouts. The safety performance of multilane roundabouts appears to be especially sensitive to design details, such as lane width.

Rodegerdts et al. (*70*) further concluded that drivers at roundabouts in the United States currently appear to be somewhat tentative, using roundabouts less efficiently than models suggest is the case in other countries around the world. In addition, the number of lanes has a clear effect on the capacity of a roundabout entry; however, the fine details of geometric design—lane width, for example—appear to be secondary and less significant than variations in driver behavior at a given site and between sites. Although the project was unable to establish a strong statistical relationship between speed and safety, the importance of controlling speed in roundabout design is well established internationally. Anecdotal evidence suggests the importance of considering design details in multilane roundabout design, including vehicle path alignment, lane widths, and positive guidance to drivers through the use of lane markings.

SUMMARY OF LITERATURE

Table 37 presents a summary of the left-turn lane design guidelines from the literature.

Table 37. Summary of left-turn lane design guidelines from the literature.

Source	Queue Storage Length	Entering Taper Length	Deceler-ation Length	Width	Offset	Dual Turn Lanes	Innovative Intersection Design
Green Book (*5*)	✓	✓	✓	✓			
Bonneson and Fontaine (*12*)	✓		✓				
Harmelink (*1*)	✓						
Kikuchi et al. (*60*)	✓	✓				✓	
Gard (*61*)	✓						
Lertworawanich and Elefteriadou (*62*)	✓						
Fitzpatrick et al. (*63*)	✓	✓	✓	✓	✓	✓	
Connecticut Department of Transportation (*64*)					✓		
Staplin et al. (65)					✓		
Tarawneh and McCoy (*66*)					✓		
McCoy et al. (*67*)					✓		
Yan and Radwan (*68*)					✓		
Jagannathan and Bared (*69*)							✓
Rodegerdts et al. (*70*)							✓

CHAPTER 4

DRIVER BEHAVIOR STUDY

BACKGROUND

While many intersections in urban environments are signalized, numerous intersections are unsignalized and have considerable left-turn activity. Particularly in areas with dense development, left-turn movements can be problematic because a wide variety of activity (through traffic, pedestrian traffic, parking maneuvers, etc.) usually occurs in a confined space and competes for the driver's attention. The confined space in many dense urban developments also generally prohibits the addition of left-turn lanes where none currently exist; in other locations, the road surface is restriped to create a left-turn lane alongside narrower through lanes. As a result, left turns in dense urban environments take place in conditions where drivers must consider many factors in making the decision to complete a left-turn maneuver; drivers' responses to these factors are reflected in their behavior.

Some of these behavioral evidences (e.g., positioning in the lane, accepted gap, time to complete the turn, time spent waiting to turn at the head of the queue, etc.) can be measured and subsequently analyzed for patterns and trends. For this project, videotaped observations recorded vehicle and pedestrian operational characteristics. Video recording permits review and data reduction after the actual crossing event occurs—a valuable approach when trying to measure turning events or gaps accepted.

STUDY OBJECTIVES

The objectives of the driver behavior study were to:

- Obtain needed data for calibrating the simulation model and
- Obtain data to update the Harmelink (*1*) procedure.

STUDY LOCATION

To conduct this research, left-turn movements were studied at 30 sites that were located in the following metropolitan areas:

- College Station/Bryan, Texas;
- Houston, Texas;
- Staten Island, New York; and
- Phoenix, Arizona.

The sites were selected based on a variety of intersection arrangements and geometric characteristics, including:

- Number of lanes on the major roadway—two or four lanes;
- Presence of a left-turn lane—yes or no;

- Signal coordination—location is near enough to a signal to be affected or far enough from a signal to result in random arrival; and
- Approach speed range—low or high speed, with posted speed limits between 25 and 40 mph being defined as low speed and posted speed limits of 45 mph or more being defined as high speed.

Table 38 lists the 30 sites used in this study and their corresponding geometric characteristics.

DATA COLLECTION

At the Texas sites, the data were collected through the use of video cameras mounted on a data collection trailer. The cameras were raised approximately 30 ft high to record a bird's-eye view of the study area. Figure 13(a) shows the trailer with the mast arm extended at one of the sites. At the Arizona and New York sites, camcorders were used. Figure 13(b) illustrates one installation.

The video recorded the movements at the intersection for at least 4 hours, observing the advancing/opposing traffic and left-turn movement. A time stamp was imprinted on the video so that the precise times of each turning movement could be reduced from the video. In addition to the video, site-specific data about the intersection were collected, including the geometric characteristics and measurements as well as detailed photographs of the intersection.

DATA REDUCTION

Following the data collection, each site's data were reduced to obtain the necessary information for the analysis. The reduction process began with reviewing the site video and obtaining turning movement counts using 5-minute intervals. The goal for each site was to obtain data for a minimum of 100 left-turning vehicles whose drivers had to make a decision based on the available gaps in the opposing traffic. In most cases, a 1-hour time interval provided the desired sample size. However, some sites did not have 100 left-turning vehicles within the 1-hour timeframe; therefore, additional hours were reduced for those sites. The turning movement counts were reviewed to determine which time period to reduce. Within that time period, actions of interest for each left-turning vehicle and opposing vehicle were recorded.

In addition to the time of arrival of the opposing vehicles, the following times for each left-turning vehicle were recorded:

- Time at the back of the queue,
- Time at the front of the queue,
- Time at the start of the left-turn maneuver,
- Time to clear the approaching lane,
- Time to clear the median and/or median lane (where applicable),
- Time to clear opposing lane 1, and
- Time to clear opposing lane 2 (where applicable).

Table 38. Site characteristics.

Site	Legs	Left-Turn Lane?	Median	Opposing Crossing Width (ft)	Opposing Lanes	Posted Speed Limit (mph)	Signal Density[a]
AZ-01	4	Yes	Flush	22.00	2	40	High
AZ-02	3	Yes	Flush	23.00	2	45	Low
AZ-03	3	No	None	24.00	2	40	Low
AZ-04	3	Yes	Flush	17.00	1	25	Low
AZ-05	3	No	None	24.00	2	40	Medium
AZ-06	3	No	None	20.00	1	30	High
AZ-07	4	No	None	22.00	1	25	High
AZ-08	3	Yes	Flush	19.00	1	35	High
AZ-09	4	Yes	Flush	25.00	2	40	Low
AZ-10	4	No	None	25.00	1	25	Medium
AZ-11	3	Yes	Flush	22.00	1	35	Medium
AZ-12	3	Yes	Flush	22.00	2	40	High
AZ-14	3	Yes	Flush	22.00	2	50	Low
AZ-15	3	Yes	Flush	24.00	2	55	Low
NY-01	3	No	None	21.00	2	30	Medium
NY-02	3	No	Raised	20.50	2	30	High
NY-03	4	No	None	19.67	1	30	High
NY-04	3	Yes	Raised	27.00	2	40	High
TX-01	3	Yes	Flush	11.00	1	45	Low
TX-02	4	No	None	21.00	1	65	Low
TX-03	3	No	None	22.00	2	30	Medium
TX-04	3	No	None	23.50	2	35	Medium
TX-05	3	Yes	Raised	24.00	2	45	Low
TX-06	3	No	Flush	23.00	2	45	Medium
TX-07	3	No	None	22.33	1	60	Low
TX-08	4	Yes	Flush	12.00	1	40	Medium
TX-09	4	Yes	Raised	24.00	2	40	Low
TX-10	4	Yes	Flush	23.00	2	40	Medium
TX-11	3	Yes	Raised	26.00	2	55	High

[a] Signal density:

- High = more than 2 signals/mile
- Medium = between 1 and 2 signals/mile
- Low = less than 1 signal/mile

(a) Elevated video cameras **(b) Ground-level video cameras**

Figure 13. Examples of equipment used for data collection.

RESULTS

To apply Harmelink's procedure and determine left-turn lane warrants, the following variables are needed:

- Time required for a left-turning vehicle to clear the advancing stream,
- Time to complete a left turn, and
- Critical gap.

Average Time-to-Clear or Time-to-Turn Values

A total of 3570 vehicles were observed in the field studies. Heavy trucks only represented a small portion of the data collected (39 vehicles), and since their operations are known to be slower than passenger cars, they were excluded from the evaluations. Of the remaining vehicles, 2945 vehicles started from a stopped position. A vehicle was included as starting from a stopped position if the vehicle spent at least 0.25 sec between arriving at the front of the queue and starting the left-turn maneuver. Table 39 lists the time-to-clear values for passenger cars for each site. In addition to calculating the average value, the average plus one standard deviation was also calculated, which allows the left-turn lane warrants to be designed for a greater portion of drivers, as opposed to only half of the drivers. The Harmelink procedure used average time to clear, or turning times.

Table 39. Time-to-clear and time-to-turn values for passenger cars starting from a stopped position at each site.

Site	Number of Vehicles	Average (sec)			Average + Standard Deviation (sec)		
		Clear Approach Lane	Clear Opposing Lane	Time to Turn	Time to Clear Approach[1]	Time to Clear Opposing[2]	Time to Turn
AZ-01	100	1.01	2.38	3.38	1.39	3.00	4.35
AZ-02	77	0.79	2.14	2.93	1.12	2.70	3.79
AZ-03	69	0.91	2.47	3.38	1.21	2.95	4.11
AZ-04	155	1.34	3.08	4.42	1.74	3.74	5.37
AZ-05	85	1.24	3.15	4.40	1.68	3.90	5.52
AZ-06	15	1.30	3.42	4.72	1.94	4.43	6.08
AZ-07	65	0.83	2.88	3.71	1.26	3.55	4.68
AZ-08	92	0.94	2.06	3.00	1.39	2.81	4.12
AZ-09	96	0.89	2.37	3.27	1.24	2.90	4.10
AZ-10	40	2.21	4.45	6.66	3.06	5.61	8.62
AZ-11	80	1.42	3.26	4.68	1.88	4.00	5.82
AZ-12	100	1.19	2.90	4.09	1.67	3.60	5.21
AZ-13	40	1.32	2.92	4.24	2.06	4.01	6.05
AZ-14	30	1.68	3.40	5.08	2.42	4.22	6.55
AZ-15	286	0.95	2.78	3.74	1.34	3.48	4.76
NY-01	100	0.80	2.12	2.93	1.14	2.68	3.76
NY-02	89	0.86	2.23	3.09	1.34	3.00	4.29
NY-03	85	1.01	2.53	3.53	1.36	3.20	4.52
NY-04	95	0.95	2.75	3.70	1.34	3.37	4.65
TX-01	219	0.99	2.01	3.00	1.31	2.51	3.79
TX-02	340	0.98	2.14	3.11	1.39	2.77	4.11
TX-03	113	0.80	2.28	3.08	1.10	2.78	3.79
TX-04	89	1.02	2.46	3.47	1.48	3.19	4.61
TX-05	120	1.24	2.56	3.80	1.67	3.07	4.72
TX-06	100	0.78	2.43	3.21	1.11	2.99	4.06
TX-07	32	1.17	2.20	3.36	1.65	2.81	4.41
TX-08	58	1.57	2.46	4.03	2.05	2.97	5.01
TX-09	42	2.05	3.49	5.54	2.96	4.50	7.45
TX-10	47	1.74	3.14	4.88	2.23	3.73	5.95
TX-11	86	1.99	3.79	5.78	2.67	4.75	7.39

[1] Time to clear approach = elapsed time from the start of the turning maneuver to the time when no part of the turning vehicle remains in the approach lane (e.g., rear bumper has crossed the centerline)
[2] Time to clear opposing = elapsed time from the start of the turning maneuver to the time when no part of the turning vehicle remains in the opposing lane (e.g., rear bumper has crossed the curb line extended)

Figure 14 shows each site's average turning time by crossing distance, subdivided by the number of lanes and the presence of a left-turn lane (LTL). The plot shows a general trend of longer turning times for wider crossing distances, although it also shows small turning times for some of the wider crossing distances. The effects of crossing width may not be obvious in this plot because the effects of other variables, such as posted speed limit, may be influencing the values.

Harmelink provided values for a two-lane highway and a four-lane highway (see Table 40). Table 40 compares the findings from this study with findings from previous studies. The average turning time for the sites included in this study was determined by the number of lanes and by generally assumed crossing widths for crossing one lane (12 ft or less) and crossing two lanes (22 ft or more). When subdividing by the number of lanes being crossed (see the second-from-bottom row in Table 40), the average turning time for these 30 study sites were similar—about 3.6 sec. As can be seen in Figure 14, at some sites, some vehicles crossing one lane traveled similar distances as when crossing two lanes. When only including those sites with more typical crossing widths, the average turning times are similar to the values assumed by Harmelink. For example, crossing 11 or 12 ft took an average of 3.2 sec, while Harmelink assumed 3.0 sec. Harmelink assumed a four-lane highway would have a 4.0-sec crossing time, while the sites in this study with a width greater than 22 ft took 3.7 sec. This comparison demonstrates that only including "typical" crossing widths of 12 ft for one lane and 22 ft or greater for two lanes removes several sites. Stated in another manner, several sites do not fit the "typical" lane width mode. The evaluation procedure may be better if it considers the crossing distance rather than just the number of lanes. This permits consideration of shoulders, bike lanes, and other conditions that increase the crossing distance.

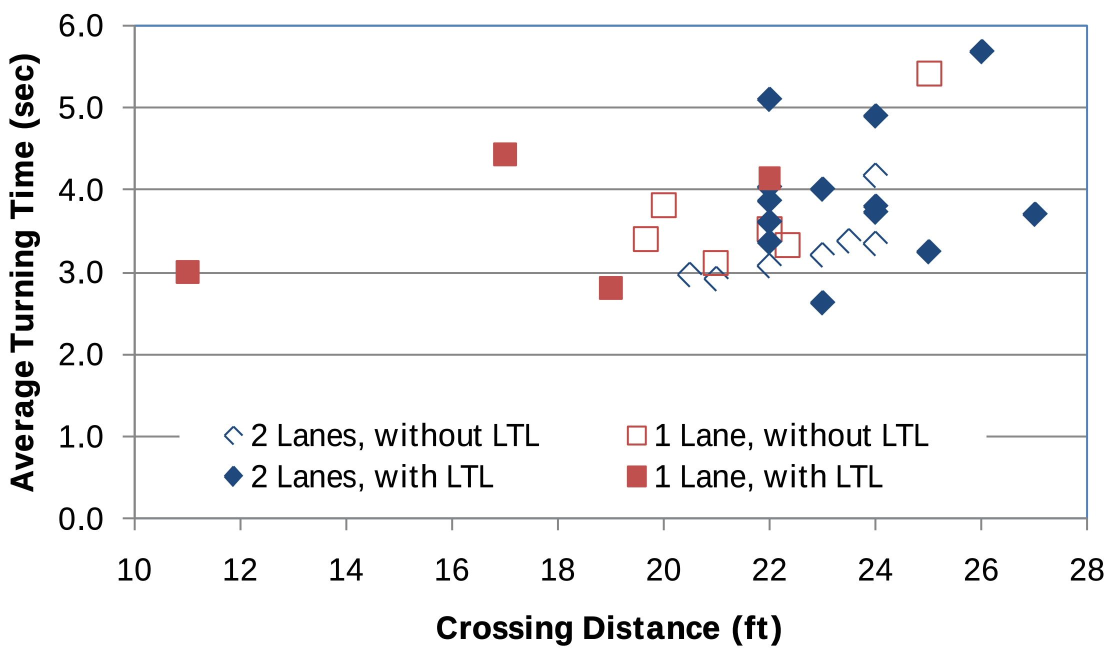


Figure 14. Plot of average turning time by crossing distance.

Table 40. Comparisons of turning times for passenger cars.

Study	Average Clear Approach Lane (sec)	Average (85th Percentile for Previous Studies or Average + Standard Deviation for Current Study) Turning Time (sec)	
		Two-Lane Highway	Four-Lane Highway
Harmelink (*1*)	1.9 sec (based on 150 vehicles)	3.0 sec	4.0 sec
1994 *Green Book* (*7*), Figure IX-33	Not calculated	4.3 sec (passenger car, based on 47-ft path)	
Miscky and Mason (*71*), 2 Pennsylvania Intersections	Not provided	4.0 (4.6) sec 4.3 (5.1) sec	Not studied
Fitzpatrick and Wolff (*13*), 1 Texas Intersection	Not provided	3.4 (4.1) sec (based on 71 vehicles)	Not studied
Current Study (Subdivided by Number of Lanes)	1.1 sec (based on 2945 vehicles)	3.6 (4.7) sec (based on 1181 vehicles)	3.7 (5.1) sec (based on 1764 vehicles)
Current Study (Grouped by Typical Crossing Width)	1.1 sec (based on 2945 vehicles)	3.2 (4.1) sec (based on 277 vehicles, crossing width of 11 or 12 ft)	3.9 (5.4) sec (based on 1264 vehicles, crossing widths of 22 to 27 ft)
		3.6 (4.8) sec (based on 1404 vehicles, crossing width of 12.5 to 21.5 ft)	

Creating Prediction Equations

Several variables were available for investigating the influence on time-to-clear values, including variables specific to the vehicle/driver and variables associated with the intersections.

Time-to-clear values are:

- Clear approach lane (sec);
- Clear opposing lane(s) (sec); and
- Turning time, which was a sum of clear approach lane and clear opposing lane values (sec).

Vehicle/driver variables are:

- Accepted lag or gap time (sec),
- Time in the queue (sec), and
- Time at the head of the queue (sec).

Intersection variables are:

- Number of legs (four or three legs),
- Presence of a left-turn lane (yes or no),
- Median type (flush, none, or raised),

- Crossing width (ft),
- Number of lanes (crossing either one or two lanes),
- Posted speed limit (mph), and
- Signal density (high is more than two signals per mile, medium is between one and two signals per mile, and low is less than one signal per mile).

Over 3500 driver maneuvers were available for the investigation. The following discussion focuses on the turning time evaluation.

Analysis of the time-to-clear values began with considering which variables to include in the models. Initially, the number of lanes was believed to be correlated with crossing width, and therefore both variables would not be included in the same model. Additional investigations revealed that when the left-turning vehicle is crossing one lane, it is crossing between 11 and 25 ft of pavement. When the vehicle is crossing two lanes, it is crossing between 20.5 and 27 ft of pavement. Therefore, there is some overlap in crossing distance values. Several of the sites with one lane had a parking lane or bike lane, which can reinforce the impression that the site is a lower-speed facility. Therefore, both variables were considered in the models.

Because both random effects from drivers and fixed effects from the intersection characteristics are present, a linear mixed-effects model was used. Several combinations of variables were considered. A log transformation was used with the accepted gap/lag time, the head of the queue time, and the time in the queue. Posted speed limit was tried as both a continuous variable and as a discrete variable. The presence of a left-turn lane, signal density group (low, medium, or high), and number of lanes were all discrete variables. The Akaike Information Criterion (AIC) and Bayesian Information Criterion (BIC) were used to select the better models. Table 41 shows the model selected for turning time.

To provide an appreciation for the impacts the variables have on predicting turning time, a reasonable range for each variable was selected. This range was used in the equations to illustrate each variable's contribution to the total turning time. Figure 15 lists the range selected for low, middle, and high and shows the plots of the results. The x-axis uses 1 to represent low, 3 to represent middle, and 5 to represent high values. Those variables that plot close to 0 (y-axis) have minimum influence on predicting time to turn and include lag/gap time, time at the head of the queue, time in the queue, and the number of lanes. Crossing width (shown with a triangle) followed by posted speed limit (shown with open squares) showed the greatest influences. The continuous lines without markers show the intercept or the predicted time to turn using the assumed values listed in the figure. Figure 16 shows the results by crossing width when using a middle value for gap (7.5 sec), time at the head of the queue (7.0 sec), time in the queue (30.0 sec), and 45-mph posted speed limit. When the crossing width is 26 ft, Figure 16 shows that it takes 4.8 sec to cross a two-lane roadway or 5.3 sec to cross a four-lane roadway. Figure 17 shows the results by posted speed limit when assuming two lanes. If the crossing width is 17 ft, then the predicted turning time is 1.9 sec for 65 mph, 2.3 sec for 55 mph, 2.8 sec for 45 mph, 3.2 sec for 35 mph, and 3.7 sec for 25 mph.

Table 41. Regression model for turning time.

Linear mixed-effects model fit by maximum likelihood

AIC	BIC	logLik
10220	10337	−5091

Random effects:

Formula: ~log(LGT) + log(TAH) + log(TIQ) | Site (14)

Structure: General positive-definite, Log-Cholesky parametrization

	StdDev	Corr		
Intercept	0.6946100	(Intr)	(LGT)	(TAH)
Log(LGT)	0.1229598	0.330		
Log(TAH)	0.1756780	0.403	−0.557	
Log(TIQ)	0.1039284	0.839	−0.067	0.440
Residual	0.9942578			

Fixed effects: TT ~ log(LGT) + log(TAH) + log(TIQ) + CW + CW_SQ + PSL + NOL (15)

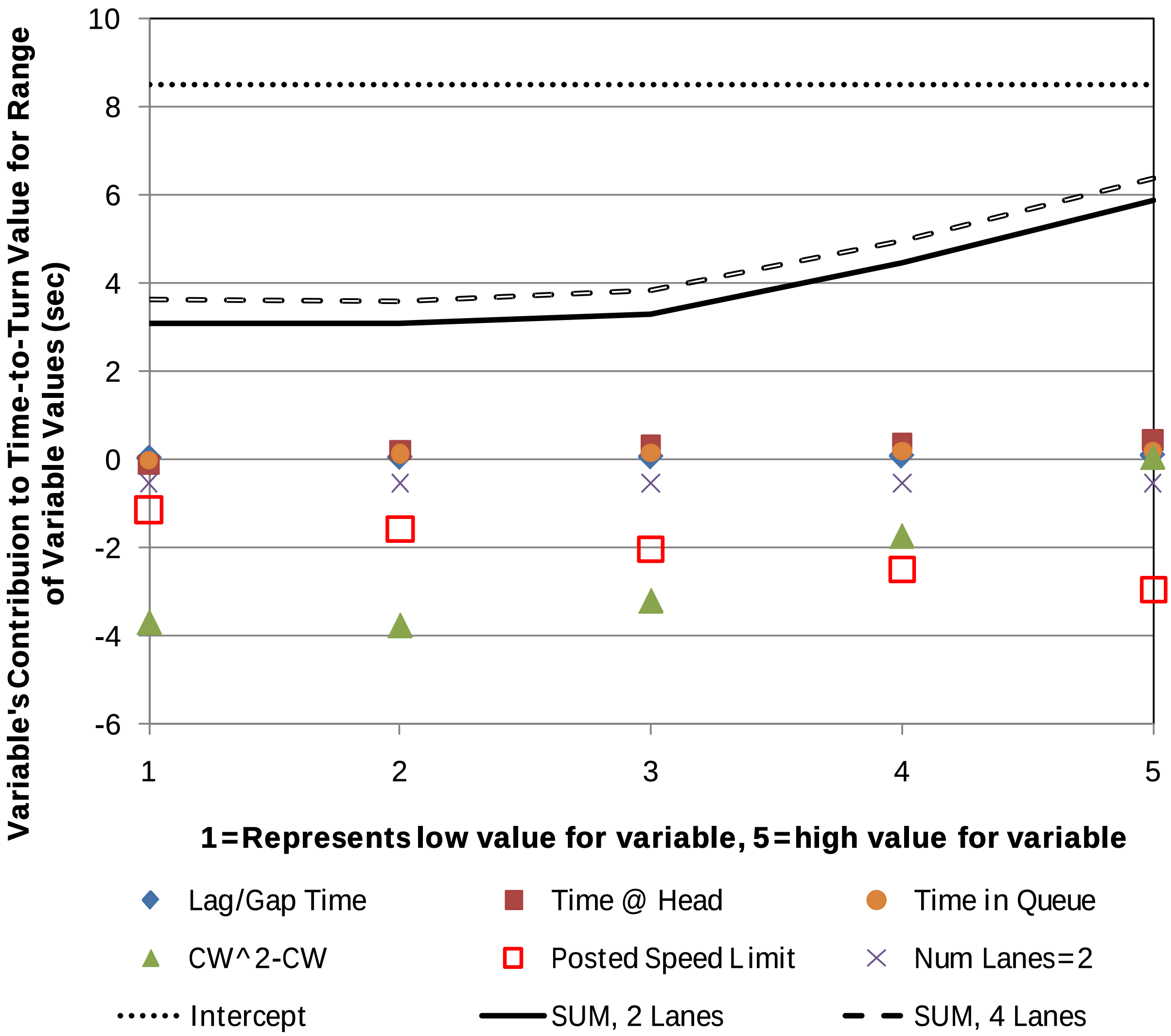
	Value	Std. Error	DF	t-value	p-value
Intercept	8.500324	2.0449751	3498	4.156688	0.0000
Log(LGT)	0.079616	0.0312464	3498	2.547988	0.0109
Log(TAH)	0.376136	0.0353656	3498	10.635661	0.0000
Log(TIQ)	0.115794	0.0255075	3498	4.539611	0.0000
CW	−0.511235	0.2145448	25	−2.382880	0.0251
CW_SQ	0.017133	0.0057080	25	3.001602	0.0060
PSL	−0.045157	0.0097226	25	−4.644551	0.0001
NOL2	−0.525688	0.2570695	25	−2.044925	0.0515

Where:

- TT = turning time (sec),
- LGT = accepted lag or gap time (sec),
- TAH = time at the head of the queue (sec),
- TIQ = time in the queue (sec),
- CW = crossing width (ft),
- CW_SQ = square of crossing width (ft),
- PSL = posted speed limit (mph), and
- NOL2 = inclusion of the indicator variable when the number of lanes is 2.

Equation:

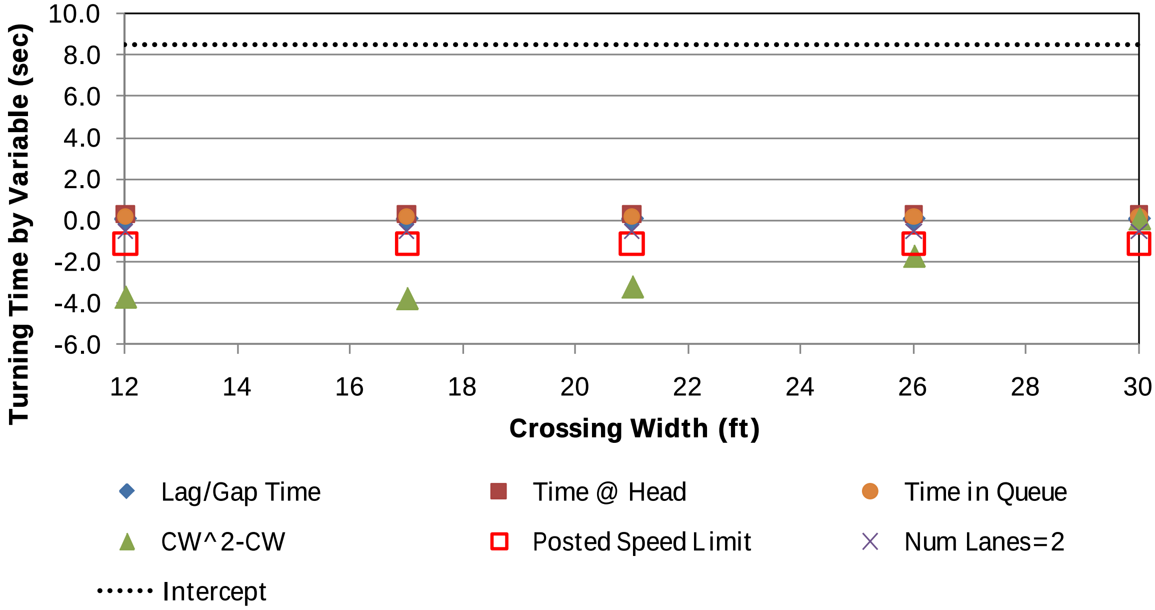
$$TT = 8.500324 + 0.017133 \times CW^2 - 0.511235 \times CW + 0.079616 \times \log(LGT) + 0.376136 \times \log(TAH) + 0.115794 \times \log(TIQ) - 0.045157 \times PSL - 0.525688 \times NOL2 \quad (16)$$



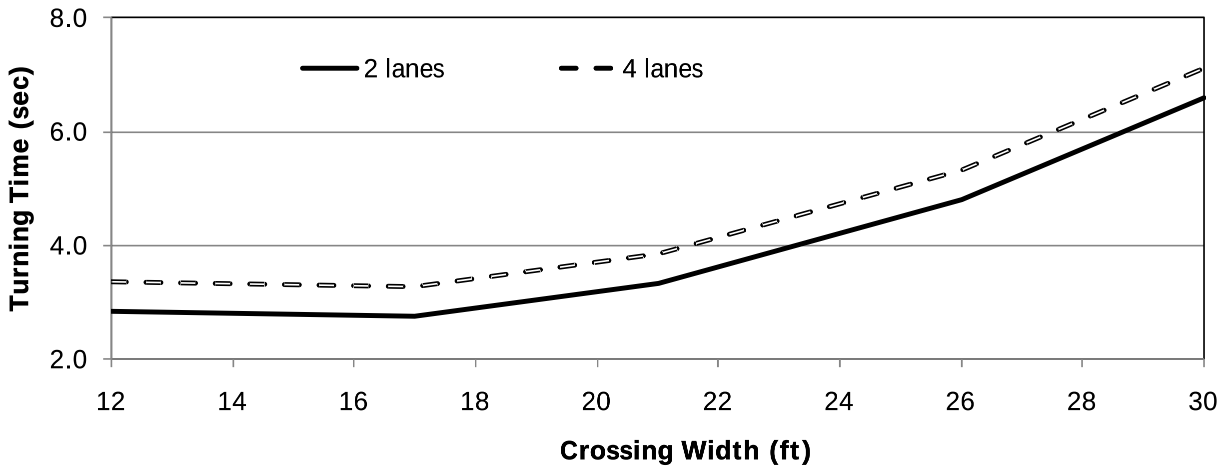
Variables	Low Value	Middle Value	High Value
Accepted Lag/Gap Time (sec)	3.0 sec	7.5 sec	20.0 sec
Time @ Head of Queue (sec)	0.5 sec	7.0 sec	15.0 sec
Time in Queue (sec)	1.0 sec	30.0 sec	60.0 sec
Crossing Width* (ft)	12 ft	21 ft	30 ft
Posted Speed Limit (mph)	25 mph	45 mph	65 mph
Number of Lanes Crossed Is 2	Yes	Yes	Yes

*considers both crossing width squared and crossing width: CW^2-CS

Figure 15. Variable's contribution to predicting turning time using low, middle, and high values.



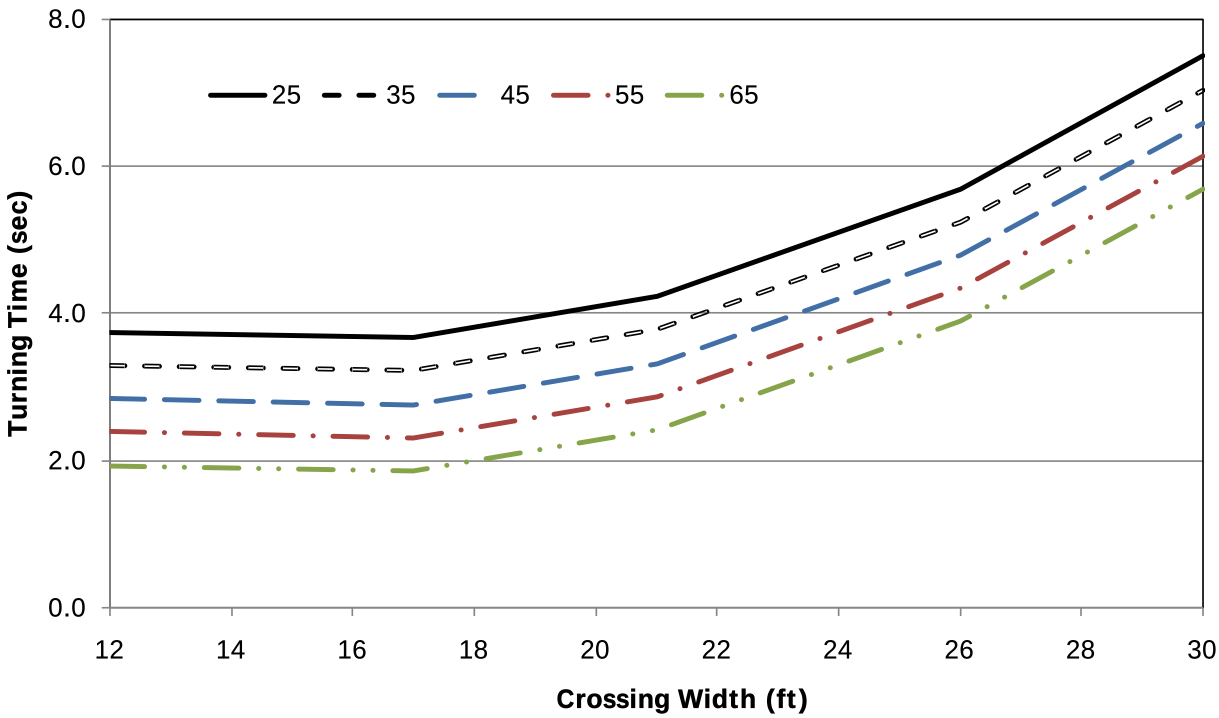
(a) Contribution of Variable to Turning Time



(b) By Number of Lanes

Variables	Assumed Value
Accepted Log/Gap Time (sec)	7.5 sec
Time @ Head of Queue (sec)	7.0 sec
Time in Queue (sec)	30.0 sec
Crossing Width (ft)	12 to 30 ft
Posted Speed Limit (mph)	45 mph

Figure 16. Variables' contribution to predicting turning time by crossing width.



Variables	Assumed Value
Accepted Log/Gap Time (sec)	7.5 sec
Time @ Head of Queue (sec)	7.0 sec
Time in Queue (sec)	30.0 sec
Crossing Width (ft)	12 to 30 ft
Number of Lanes Crossed Is 2	Yes

Figure 17. Turning time by posted speed limit (assumed two-lane highway).

The most influential variables are crossing width and posted speed limit. A change from a crossing width of 11 ft to 27 ft results in an additional 2.24 sec in total turning time when other variables are set to the middle values listed in Figure 15.

The posted speed limit variable is associated with a decrease in turning time for the higher speeds. For turning time, an additional 1.81 sec is subtracted when the roadway has a 65-mph posted speed as compared to a 25-mph posted speed limit. The posted speed limit range represented in the dataset is 25 to 65 mph.

The amount of time spent at the head of the queue had a statistically significant impact on turning time. Initial expectation was that as drivers have to wait for an acceptable gap, they drive faster to clear the roadway. However, the relationship was in the opposite direction. As the amount of time at the head of the queue increases, the clearance times also increase, but only by a small amount. A representative range observed for the 30 sites was between 1 sec and 15 sec at the head of the queue. A change from a 1-sec to 15-sec time at the head of the queue value results in an additional 0.44 sec in total turning time when other variables are set to the middle values listed in Figure 15.

A similar condition existed for time in the queue. Expectations were that drivers would drive faster after waiting in a queue. The findings were that drivers took a slightly greater amount of time after waiting in a long queue; however, the increase was very small. A representative range observed for the 30 sites was between 1 and 60 sec at the head of the queue. A change from 1 sec to 60 sec in the time at the head of the queue value results in an additional 0.21 sec in total turning time when other variables are set to the middle values listed in Figure 15.

The relationship between clearance time and the accepted lag or gap time was in the direction expected although it did not have as large an influence as initially thought. The preliminary thought was that drivers notably drive faster (i.e., lower turning time) when accepting a small gap. The evaluation only found a small increase in clearance time for larger gaps. For a 20-sec gap as compared to a 5-sec gap, the increase in turning time was about 0.05 sec.

The difference in clearance time for the number of lanes was also small. Initially it was believed that the number of lanes variable should not be included due to anticipated correlation with crossing width. When verified that it could be included, the expectation was that longer crossing times would be associated with the higher number of lanes. The finding, however, is that the time is decreased by a constant when two lanes are being crossed. For turning time, the amount is about 0.43 sec. In most situations, clearing two lanes was also accompanied by a wider crossing width. The larger time calculated due to the wider crossing width offsets some of the negative time that is added when crossing two lanes. The number of lane variable may be a surrogate for other characteristics of a multilane facility that were not captured in the other variables.

The results from the regression models can be used to calculate the turning time for use in Harmelink or other models. The suggested equation is listed in Table 41.

Critical Gap

Critical gap is defined as the time interval between two opposing vehicles that is necessary for a left-turning vehicle to safely complete a left-turn maneuver. Two methods were used to determine critical gap: logistic regression and Raff/Hart.

Logistic regression is appropriate when the dependent variable is binary or dichotomous (e.g., either the acceptance or rejection of a gap). The mean response is a probability when the dependent variable is a 0 or 1 (accept or reject) indicator variable. The shape of the response function is curvilinear and can be approximated using a logistic function. A property of a logistic function is that it can be easily converted into a linear form. The transformation is called the logistic, or logit, transformation. The simple, dichotomous choice logistic function is:

$$P(x) = \frac{1}{1+\exp\left[-(\beta_0+\beta_1 x)\right]} \tag{17}$$

Where:

$P(x)$ = probability of accepting a gap at x;
x = value related to the gap acceptance decision, gap length; and

β_o, β_1 = regression coefficients.

The logistic function can be converted to a linear form with the following transformation:

$$g\ (x) = log_e \frac{P(x)}{1-P(x)} = (\beta_0 + \beta_1 x) \qquad (18)$$

Where:

g (x) = logit, transformation of probability *P(x)*.

The logistic regression coefficients for each site were determined using the method of maximum likelihood implemented in JMP (SAS product). The maximum likelihood estimates for the coefficients are listed in Table 42. Sample logistic curves for Sites AZ-01, NY-03, and TX-03 are shown in Figure 18. The time gap for a 50 percent probability can be determined by substituting 0.5 for P in the above equation and the regression coefficients for β_o and β_1. For example, 50 percent of the drivers at TX-03 accepted a 5.02-sec gap, while drivers at AZ-01 accepted a 4.18-sec gap. The following illustrates the calculation of the 50th percentile value for TX-03:

$$g\ (x) = log_e \frac{0.5}{1-0.5} = (-4.22079 + 0.840289x) \qquad (19)$$

$$0 = (-4.22079 + 0.840289x)$$

x = 5.02 sec

Raff and Hart (*72*) defined the critical lag, L, as the size lag for which the number of accepted lags shorter than L is the same as the number of rejected lags longer than L. Raff and Hart did not include gaps in the study, arguing that one driver will only accept a gap of a particular size, but another driver may reject several gaps of the same size. More recent studies indicate that the acceptance of lags is not significantly different from the acceptance of gaps and that the lag and gap data can be combined (*73, 74*). Therefore, the lag and gap data for each vehicle maneuver were merged in this study.

Raff and Hart used the number of accepted gaps and the number of rejected gaps. The same approach could be used with cumulative percent. Both approaches were used with this dataset with similar results. An example of the Raff/Hart graphical method is illustrated in Figure 19.

Table 42. Logistic regression coefficients for field sites.

Site	β_o	β_1	Calculated Gaps (sec) Using Regression Coefficients	
			50th Percentile	85th Percentile
AZ-01	−7.08753	1.696152	4.18	5.20
AZ-02	−1.77842	0.851137	2.09	4.13
AZ-03	−2.69889	0.81338	3.32	5.45
AZ-04	−10.3758	1.654808	6.27	7.32
AZ-05	−1.81918	0.494428	3.68	7.19
AZ-06	Model Unstable			
AZ-07	−6.94864	1.285945	5.40	6.75
AZ-08	−3.91809	0.873803	4.48	6.47
AZ-09	−3.70749	0.763491	4.86	7.13
AZ-10	−1.37498	0.533511	2.58	5.83
AZ-11	−3.12467	0.602826	5.18	8.06
AZ-12	−4.21706	0.768667	5.49	7.74
AZ-13	−2.29175	0.347882	6.59	11.57
AZ-14	−29.3524	6.040599	4.86	5.15
AZ-15	−4.12241	0.826803	4.99	7.08
NY-01	−3.83717	1.049497	3.66	5.31
NY-02	−5.57702	1.405221	3.97	5.20
NY-03	−4.42662	1.090183	4.06	5.65
NY-04	−4.11451	0.770901	5.34	7.59
TX-01	−5.59645	1.183262	4.73	6.20
TX-02	−4.88761	0.981591	4.98	6.75
TX-03	−4.22079	0.840289	5.02	7.09
TX-04	−4.07481	1.157537	3.52	5.02
TX-05	−3.54230	0.681631	5.20	7.74
TX-06	−4.07647	0.810744	5.03	7.17
TX-07	−5.54214	0.98852	5.61	7.36
TX-08	−4.00150	0.904033	4.43	6.35
TX-09	−4.91986	0.910026	5.41	7.31
TX-10	−2.09213	0.585126	3.58	6.54
TX-11	−2.32148	0.483307	4.80	8.39

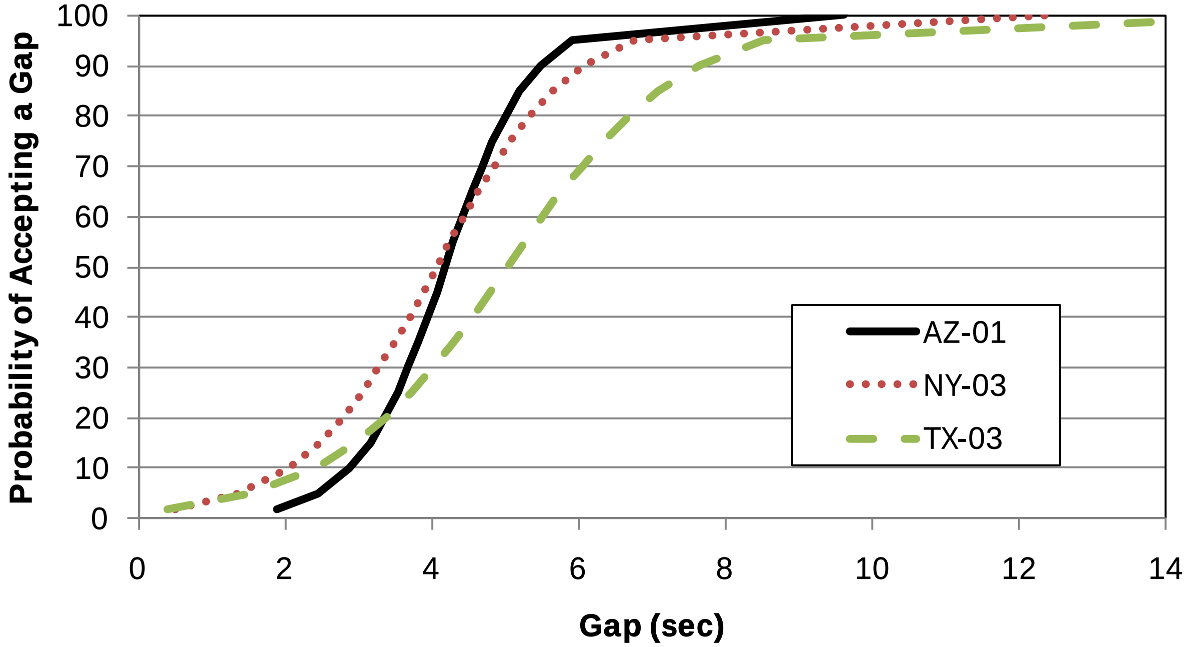
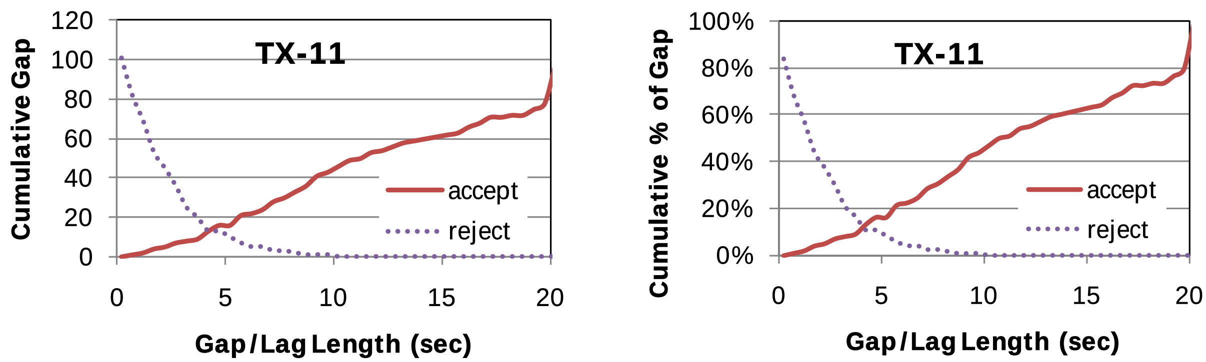


Figure 18. Plot of logit model for three field sites.



(a) Frequency **(b) Percent**

Figure 19. Illustration of Raff/Hart graphical method for identifying critical gap.

The findings from the Raff/Hart method and the logistic regression method are listed in Table 43. The sites were also grouped by crossing distance and posted speed limit to seek trends in gap acceptance values.

Previous research in simulators has found that drivers accept smaller gaps at higher speeds. Alexander et al. (*75*) used a simulator to evaluate turning left from the major road onto the minor road. The velocity of the oncoming traffic was the variable that had the greatest effect on the median accepted gap size. Yan et al. (*76*) also studied drivers' gap acceptance decision in a simulator focusing on major traffic speed, driver age, and driver gender. They found that traffic speed had a significant effect on the gap acceptance maneuver. The mean value gap found within the speed, gender, and age groups used by Yan et al. is shown in Table 44.

Table 43. Results for gap acceptance methods.

Site	Left-Turn Lane?	Posted Speed Limit (mph)	Lanes	Crossing Width (ft)	Raff/Hart Gap Value	Logit 50th Percentile	Logit 85th Percentile	Number of Cars
TX-01	Yes	45	1	11	4.82	4.73	6.20	219
TX-08	Yes	40	1	12	5.50	4.43	6.35	170
				Weighted average:	**5.12**	**4.60**	**6.26**	195
AZ-04	Yes	25	1	17	6.90	6.27	7.32	151
AZ-08	Yes	35	1	19	4.72	4.48	6.47	104
NY-03	No	30	1	19.67	4.33	4.06	5.65	94
AZ-06	No	30	1	20	3.70	Model unstable		25
				Weighted average:	**5.43**	**5.14**	**6.62**	94
NY-02	No	30	2	20.5	4.37	3.97	5.20	94
NY-01	No	30	2	21	4.31	3.66	5.31	79
TX-02	No	65	1	21	6.31	4.98	6.75	268
AZ-01	Yes	40	2	22	4.07	4.18	5.20	99
AZ-07	No	25	1	22	6.51	5.40	6.75	87
AZ-11	Yes	35	1	22	5.72	5.18	8.06	100
AZ-12	Yes	40	2	22	5.25	5.49	7.74	99
AZ-13	Yes	55	2	22	9.03	6.59	11.57	74
AZ-14	Yes	50	2	22	4.77	4.86	5.15	34
TX-03	No	30	2	22	5.19	5.02	7.09	108
TX-07	No	60	1	22.33	7.19	5.61	7.36	33
AZ-02	Yes	45	2	23	3.44	2.09	4.13	97
TX-06	No	45	2	23	6.12	5.03	7.17	100
TX-10	Yes	40	2	23	4.68	3.58	6.54	100
TX-04	No	35	2	23.5	4.27	3.52	5.02	94
AZ-10	No	25	1	25	4.85	2.58	5.83	202
				Weighted average:	**5.35**	**4.34**	**6.51**	104
AZ-03	No	40	2	24	4.07	3.32	5.45	69
AZ-05	No	40	2	24	5.67	3.68	7.19	90
AZ-15	Yes	55	2	24	5.77	4.99	7.08	277
TX-05	Yes	45	2	24	6.33	5.20	7.74	114
TX-09	Yes	40	2	24	5.81	5.41	7.31	63
AZ-09	Yes	40	2	25	5.76	4.86	7.13	98
TX-11	Yes	55	2	26	5.69	4.80	8.39	96
NY-04	Yes	40	2	27	5.85	5.34	7.59	92
				Weighted average:	**5.70**	**4.79**	**7.26**	112

Table 44. Mean values of gap (sec).

Speed	Yan et al. Simulator Study (*76*)						This Study		
	Female			Male			Number Vehicles	Raff/ Hart	Logit 50th Percentile
	Young	Middle	Old	Young	Middle	Old			
25 mph	7.56	6.97	10.99	6.35	6.60	8.76	440	5.88	4.40
55 mph	6.00	5.63	7.11	5.26	5.61	6.23	447	6.29	5.21

This field study included three sites with a 25-mph speed limit and three sites with a 55-mph speed limit. The weighted average gap accepted using the Raff/Hart method and the logit 50th percentile was calculated and is also listed in Table 44. These findings are also illustrated in Figure 20. The relationship between posted speed limits from the field studies is similar to the finding from the simulator study—smaller gaps are accepted at the higher posted speed sites. The difference is on the order of 1.0 to 1.5 sec. The average gaps accepted from the field studies were smaller than the simulator study, which could be a reflection of the more ideal situation for drivers in a simulator (e.g., lack of pressure to make a gap decision from other drivers, no need to rush to a destination for an appointment, etc.).

Gap acceptance at the field study sites was also examined by crossing distance. Logically, a driver would want a longer gap to cross additional pavement. Figure 21 shows the gap acceptance for each site by crossing distance. An apparent trend of larger gaps for greater distances or even the inverse is not evident in Figure 21. Weighted averages were calculated for four groups:

- One lane (11 to 12 ft),
- Wide one lane (17 to 20 ft),
- Narrow two lanes or very wide one lane (20.5 to 23.5 ft), and
- Two lanes or very wide one lane (24 to 27 ft).

The weighted averages are illustrated in Figure 22. The results using the Raff/Hart method or the logit 85th percentile were as expected. Gap acceptance values increase as the crossing width increases, although only by a small amount (less than 1 sec between the one-lane group and the two-lane or very-wide-one-lane group). The logit 50th percentile group showed a contrary relationship with smaller gaps for wider crossing distances.

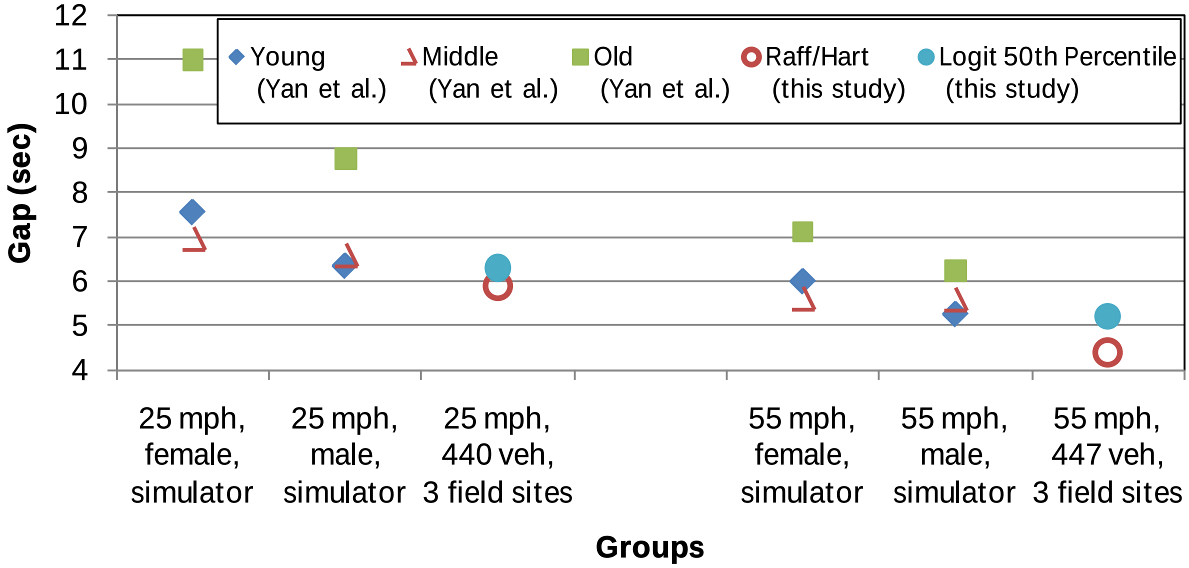


Figure 20. Plot of median gap acceptance for 25-mph and 55-mph sites from this field study and Yan et al. (76).

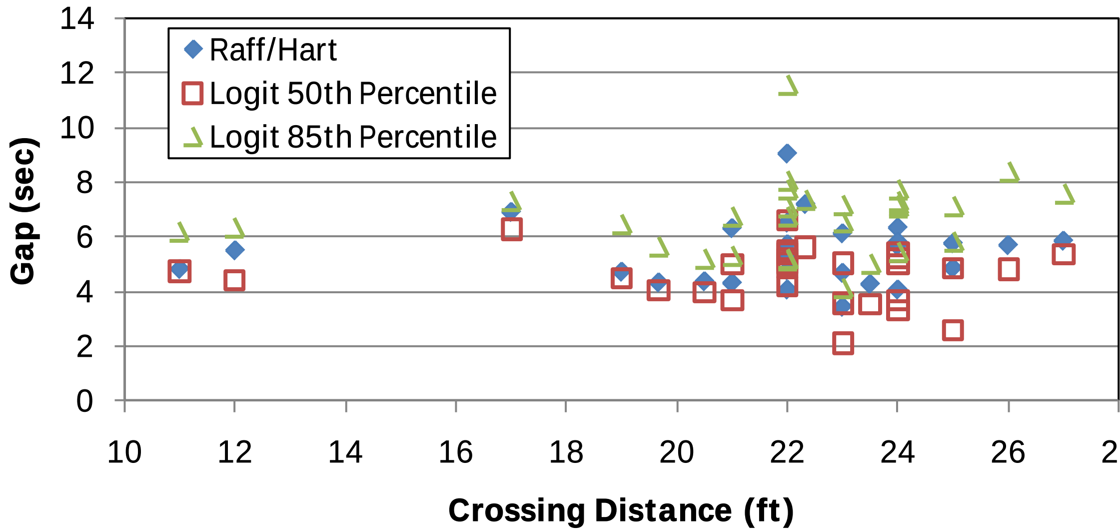


Figure 21. Gap acceptance result by crossing distance for field study sites.

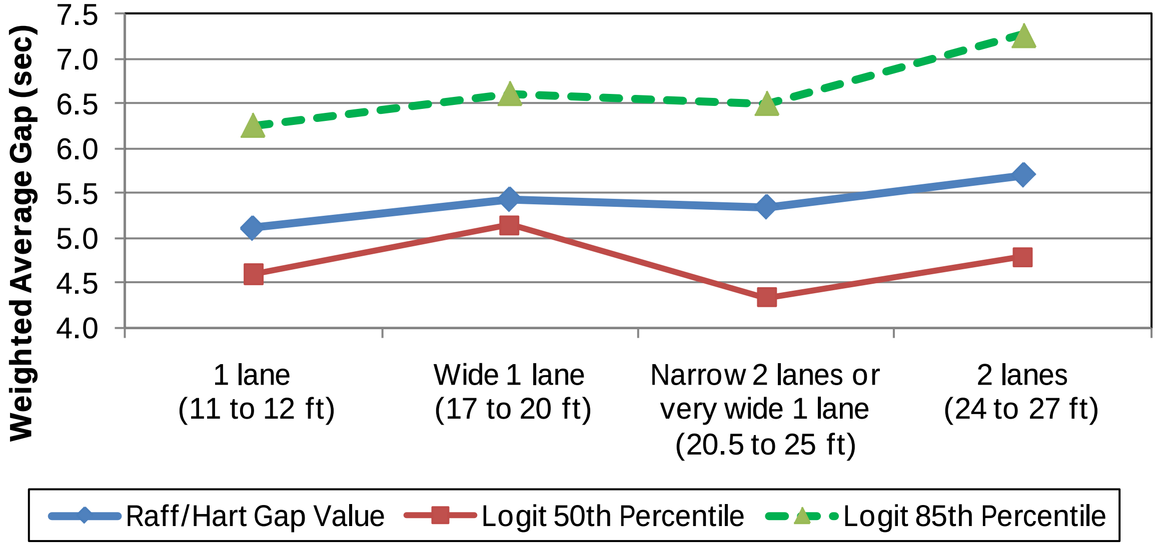


Figure 22. Gap acceptance result by crossing distance group for field study sites.

CHAPTER 5

DELAY, CRASH, AND CONSTRUCTION COST STUDIES

Left-turn lanes can provide benefits in safety as well as operations. Left-turn lanes can reduce the potential for collisions by providing safer left-turn operations. They can also reduce delay and improve left-turn capacity by removing stopped left-turn vehicles from the main travel lane.

A benefit-cost approach to justify right-turn deceleration lanes was presented by Potts et al. (*58*). The benefits of the right-turn lane were determined for crash reduction (change in number of crashes) and delay reduction (improvements in arterial capacity from removing the slower-moving vehicles from the main traffic stream). The cost of the right-turn lane reflected construction costs.

This chapter presents a similar approach to determine when a left-turn lane would be justified. The steps include:

- Identify an economic analysis procedure,
- Use simulation to determine delay savings from installing a left-turn lane and delay increase from a new development,
- Calculate crash costs and crash reduction savings using safety performance functions and crash accident modification factors from the *Highway Safety Manual* (*77*), and
- Determine construction cost.

ECONOMIC ANALYSIS PROCEDURE

Economic analysis can provide a useful method for combining traffic operations and safety benefits of left-turn lanes to identify situations in which left-turn lanes are and are not justified economically. The following equation shows how to calculate the benefit-cost ratio:

$$\frac{B}{C} = \frac{Delay\ Reduction + Safety\ Improvements}{Construction\ Costs} \tag{20}$$

When the B/C ratio exceeds 1.0, a left-turn lane is considered economically justified because the benefits are greater than the costs.

The following equations provide an overview of the components needed for the evaluation of left-turn lanes:

$$\frac{B}{C} = \frac{\left[\{DR(V_{TR}, V_{LT}, Hr) \times DC\} + \left\{\langle N_{spf}\left(ADT_{maj}, ADT_{min}\right)(1 - AMF_{LT})\rangle \times AC\right\}\right] \times USPWF(i,n)}{CC} \tag{21}$$

$$USPWF(i,n) = \frac{(1+i)^n - 1}{i(1+i)^n} \tag{22}$$

Where:

B/C	=	benefit-cost ratio;
$DR(V_{TR}, V_{LT}, Hr)$	=	delay reduction (veh-hr/year) from left-turn lane installation as a function of V_{TR}, V_{LT}, Hr;
V_{TR}	=	major-road volume during the traffic period of interest (veh/hr);
V_{LT}	=	left-turn volume during the traffic period of interest (veh/hr);
Hr	=	number of hours in a year for the traffic period of interest (hr);
DC	=	user cost savings from delay reduction ($/veh-hr);
$N_{spf}(ADT_{maj}, ADT_{min})$	=	safety performance function to estimate the intersection-related predicted average crash frequency for base conditions;
AMF_{LT}	=	accident modification factor for installation of a left-turn lane;
AC	=	user cost savings from crash reduction ($/crash);
$USPWF(i,n)$	=	uniform series present worth factor as a function of i and n;
I	=	minimum attractive rate of return expressed as a decimal (i.e., 0.04 for a 4 percent return);
n	=	number of years; and
CC	=	estimated construction cost for a left-turn lane ($).

The B/C ratio is composed of two major components:

- Benefit in crash reduction and
- Benefit in reduction of travel time delays.

Both of these terms are multiplied by the uniform series present worth factor to convert them from annual benefit amounts to the present value of a time series of annual benefits.

DELAY

Scenarios

Computer simulation was used to evaluate the operational benefits of a left-turn lane on intersection delay at an unsignalized intersection. The purpose of the effort was to assess the delay savings from installing a left-turn lane.

The key operational issue related to left-turn lanes at unsignalized intersections or driveways is the operational delay to traffic on the major street. If there is substantial delay to through traffic caused by queued left-turn vehicle(s), provision of a left-turn lane can reduce that delay.

Two scenarios were explored:

- Existing site and
- New development.

In the existing site scenario, the comparison identifies the benefits when the left turns at an existing driveway or intersection are provided a left-turn lane. The total average delay for when a left-turn lane is present is subtracted from the total average delay when a left-turn lane is not present. This difference represents the total average delay savings per vehicle at the intersection on the major roadway.

In the second scenario, the baseline condition is that no delay is present at the location because there is no left-turn demand. When a new development is proposed, the added delay to the system is determined from simulation. The total average delay for the without left-turn lane scenario represents the anticipated delay being added to the system when a new development creates left-turn demand.

Simulation

To conduct the operational analysis of left-turn lanes, a microsimulation model (VISSIM) was used to measure the impact of left-turn vehicles on intersection delay. The following variables were examined:

- Presence of a left-turn lane (yes or no);
- Number of through lanes on the major street (two or four);
- Left-turn lane volume (20, 60, 100, or 140 veh/hr);
- Through traffic volume:
    - Two lanes on the major road: 400, 600, or 800 veh/hr/approach or
    - Four lanes on the major road: 400, 800, or 800 veh/hr/lane; and
- Traffic speed on the major road: 30, 40, or 50 mph.

Assumptions included:

- Arrival is random.
- The standard deviation for speeds is 5 mph.
- The critical gap for left-turning vehicles is 5 sec.
- The default traffic composition is 98 percent automobiles and 2 percent trucks.
- The unsignalized intersections and driveways considered have no traffic control requiring a stop or yield by vehicles on the major street.
- The left-turn lane, when present, is long to avoid having queue spillbacks from the lane.

A series of simulation modeling runs were conducted. To evaluate delay, each volume scenario was modeled without a left-turn lane and again with a left-turn lane. Ten runs were conducted per scenario. The system-wide measure of performance used was average delay per vehicle for the whole network, measured in seconds. To determine the benefit of adding a left-turn lane, the difference between the average total delays with and without a left-turn lane was calculated. This value represents the operational benefit of having a left-turn lane at an existing site. Figure 23 illustrates the delay reductions, while Figure 24 illustrates the added delay due to a new development. A review of the data revealed that delay:

- Increases as the number of left-turn vehicles increases,
- Increases as the volume on the major road increases,
- Decreases or is relatively constant as the speed limit increases for two-lane highways,
- Increases or is relatively constant as the speed limit increases for four-lane highways, and
- Decreases or is relatively constant as the number of lanes increases (with the exception of a 600- or 800-veh/hr/ln approach volume, 140-veh/hr left-turning volume, and 50-mph speed limit).

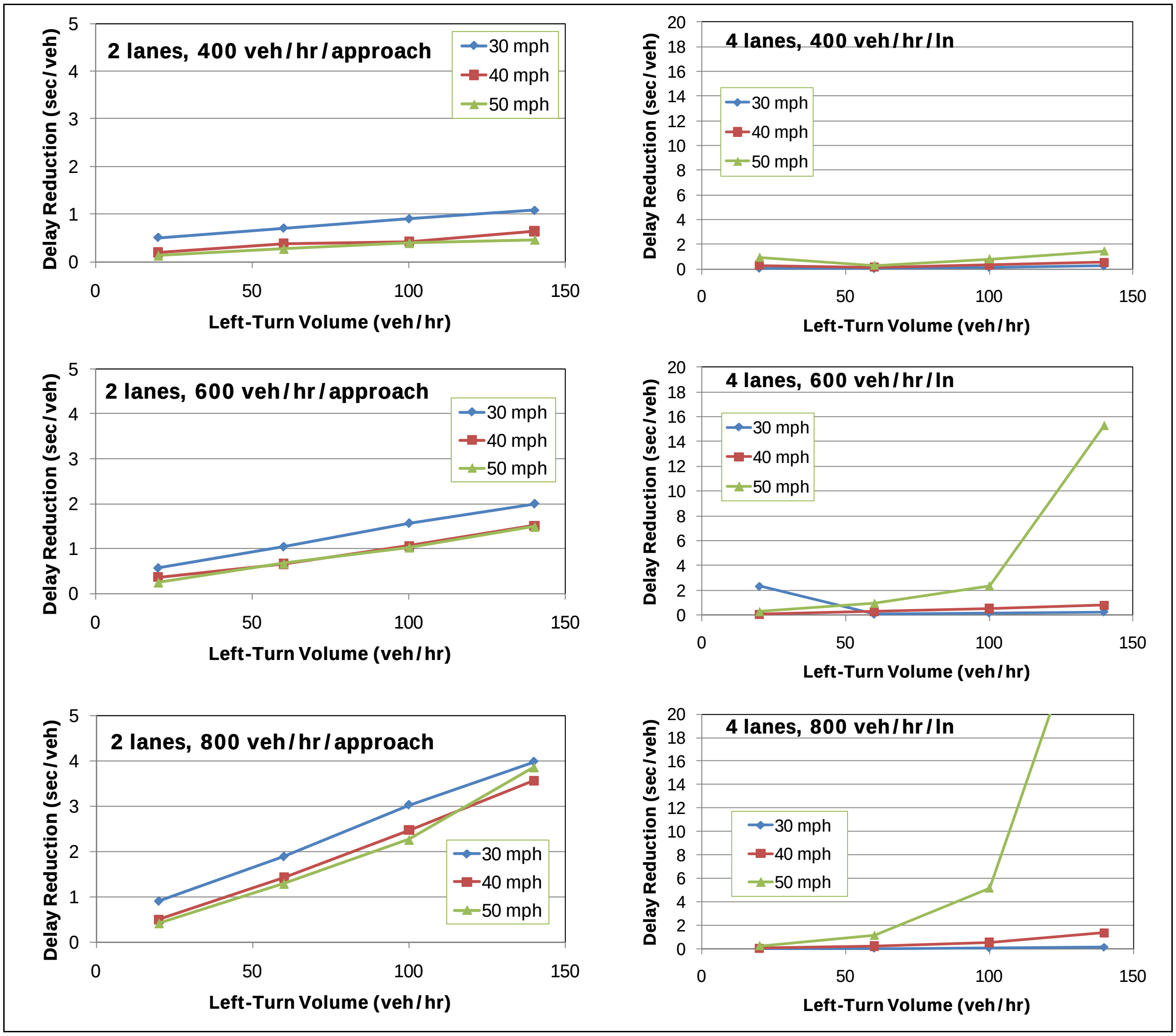


Figure 23. Simulation-estimated delay reduction when adding a left-turn lane at an existing site.

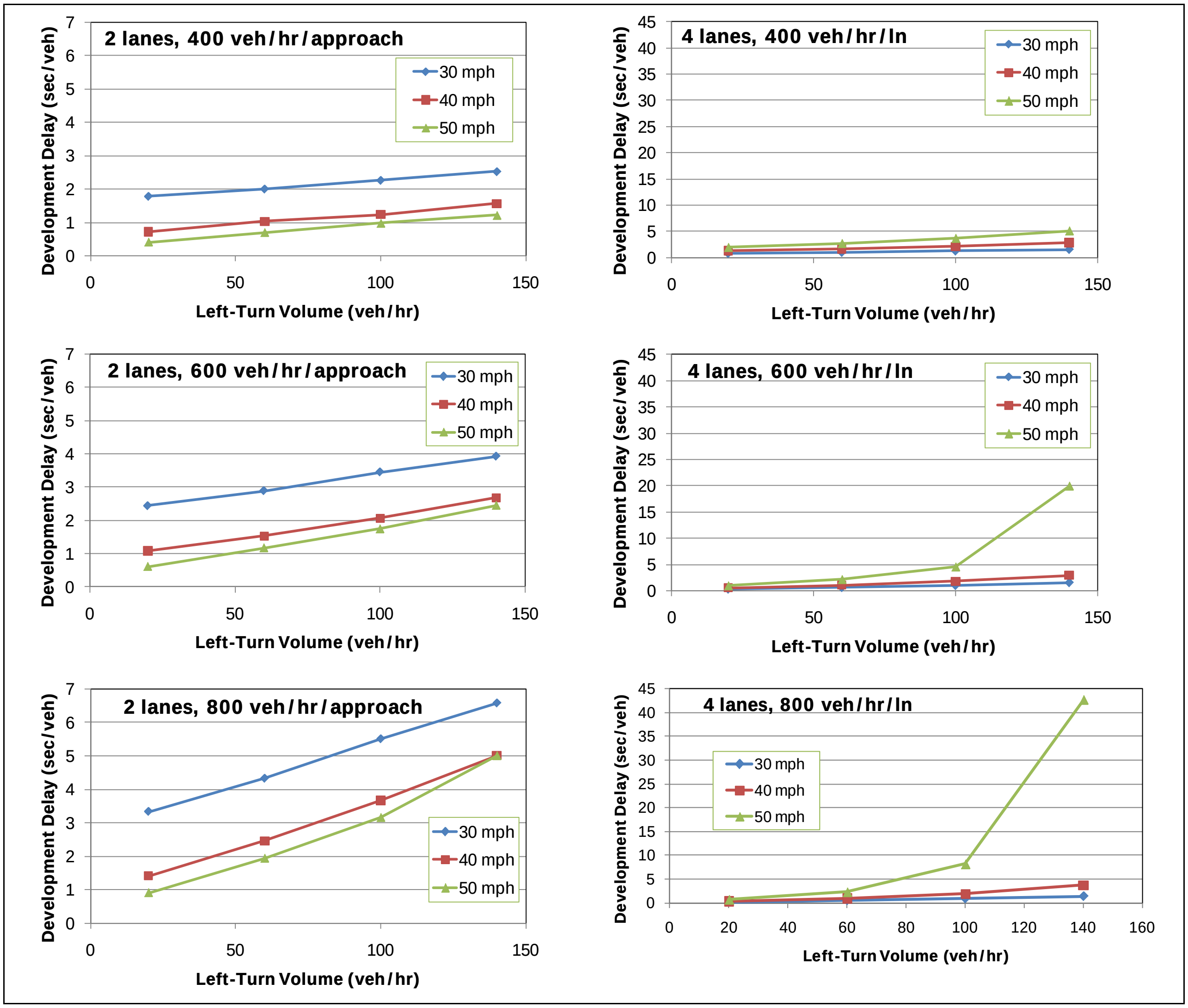


Figure 24. Simulation-estimated added delay for a new development (no left-turn lane scenario).

To facilitate the evaluations, linear regression was used to generate equations to represent the anticipated delay for different combinations of number of lanes, major-road volume, left-turn volume, and posted speed. In most cases the R^2 value was greater than 0.8, which is not unexpected since this is modeling of microsimulation results. In a few cases the linear regression results in a negative delay reduction (i.e., delay increases because of the installation of a left-turn lane). This reflects the characteristic of linear regression and/or the stochastical nature of the simulation. For those situations, a minimum delay reduction of 0.01 sec/veh was assumed. The data for four lanes, 50 mph, 140 veh/hr turning left, and 600 or 800 veh/hr/ln on the major roadway were not included in the regression because those points were considered to be outliers. Table 45 lists the intercept terms and coefficients by number of lanes and speed limit for delay reduction for existing sites and delay generated due to new developments. Table 46 lists examples of the calculated delay for the two scenarios for 20, 60, 100, and 140 left-turn vehicles.

Table 45. Regression coefficients to predict the delay determined from simulation.

Number of Lanes	Speed Limit (mph)	Delay Reduction When Adding a Left-Turn Lane to an Existing Site Coefficients			Delay Due to New Development Coefficients		
		Intercept	Major Volume	Left-Turn Volume	Intercept	Major Volume	Left-Turn Volume
2	30	−2.10008	0.00412	0.01423	−1.97700	0.00695	0.01532
	40	−2.30383	0.00395	0.01289	−2.29792	0.00499	0.01675
	50	−2.52283	0.00411	0.01373	−2.68758	0.00482	0.01860
4	30	−1.37877	0.00239	0.00766	−2.41700	0.00563	0.01490
	40	−2.34039	0.00360	0.00965	−2.90076	0.00530	0.01927
	50	−3.61735	0.00582	0.01407	−5.46192	0.00881	0.02988

Table 46. Predicted delays.

Number of Lanes	Speed Limit (mph)	Major Volume (veh/hr/ln)	Delay [a] Reduction (sec/veh) When Adding a Left-Turn Lane to an Existing Site for Left-Turn Volume (veh/hr) of:				Delay [a] (sec/veh) due to New Development for Left-Turn Volume (veh/hr) of:			
			20	60	100	140	20	60	100	140
2	30	400	0.0	0.4	1.0	1.5	1.1	1.7	2.3	2.9
		600	0.7	1.2	1.8	2.4	2.5	3.1	3.7	4.3
		800	1.5	2.0	2.6	3.2	3.9	4.5	5.1	5.7
	40	400	0.0	0.0	0.6	1.1	0.0	0.7	1.4	2.0
		600	0.3	0.8	1.4	1.9	1.0	1.7	2.4	3.0
		800	1.1	1.6	2.1	2.7	2.0	2.7	3.4	4.0
	50	400	0.0	0.0	0.5	1.0	0.0	0.4	1.1	1.8
		600	0.2	0.8	1.3	1.9	0.6	1.3	2.1	2.8
		800	1.0	1.6	2.1	2.7	1.5	2.3	3.0	3.8
4	30	400	0.0	0.0	0.3	0.6	0.1	0.7	1.3	1.9
		600	0.2	0.5	0.8	1.1	1.3	1.9	2.4	3.0
		800	0.7	1.0	1.3	1.6	2.4	3.0	3.6	4.2
	40	400	0.0	0.0	0.1	0.5	0.0	0.4	1.1	1.9
		600	0.0	0.4	0.8	1.2	0.7	1.4	2.2	3.0
		800	0.7	1.1	1.5	1.9	1.7	2.5	3.3	4.0
	50	400	0.0	0.0	0.1	0.7	0.0	0.0	1.1	2.2
		600	0.2	0.7	1.3	[b]	0.4	1.6	2.8	[b]
		800	1.3	1.9	2.4	[b]	2.2	3.4	4.6	[b]

[a] Delay is defined based upon all vehicles in the system microsimulation.
[b] Beyond limit of regression, use value scaled from Figure 23 or Figure 24.

The following is an illustration of the use of the equation for delay reduction when there are 600 veh/hr/ln on the major roadway and 60 left-turning vehicles during the peak hour:

$$DR_{2\ ln,\ 40\ mph} = -2.30383 + 0.00395\ Major_{PHV} + 0.01289\ LTL_{PHV} \tag{23}$$

$$DR_{2\ ln,\ 40\ mph} = -2.30383 + 0.00395(600) + 0.01289(60)$$

$$= 0.83957$$

Where:

$DR_{2\ ln,\ 40\ mph}$ = delay reduction when adding a left-turn lane on a two-lane highway with a 40-mph posted speed limit (sec/veh),
$Major_{PHV}$ = major-road peak-hour volume (veh/hr/ln), and
LTL_{PHV} = left-turn lane peak-hour volume (veh/hr).

The regression delay reduction equations (coefficients in Table 45) were generated to facilitate the development of benefit-cost ratios. The equations permit automating calculations within a spreadsheet, which results in the ability to test more scenarios in the determination of left-turn lane warrants.

Delay for Entire Year

The simulation provides predictions of delay per vehicle in the system. This value needs to be converted to delay at the intersection for the entire year. To perform the conversion, the assumed number of hours along with the percent of the ADT represented by each traffic period is needed. Table 47 provides the assumptions used to convert sec/vehicle delay into hours of delay for the year at the intersection.

Travel Time Delay Savings

The national congestion constants used in the *2009 Urban Mobility Report* (*78*) are shown in Table 48. The values represent 2007 dollars. The value of person time used in the *Urban Mobility Report* is based on the value of time, rather than the average or prevailing wage rate. The average cost of time was assumed to be $15.47 per person hour for 2007.

For the analyses in this report, researchers used the 2007 value of time at $15.47 as presented in the *2009 Annual Urban Mobility Report* (*78*). The 2007 value of time was adjusted using the Consumer Price Index (CPI) for 2007 and 2009 available from the U.S. Bureau of Labor Statistics (*81*). The ratio of the 2009 to 2007 CPI value is 214.537 divided by 207.342, which is 1.03. The ratio 1.03 multiplied by $15.47 gives a 2009 value of time of $16.01.

The value represents average cost of time per person. To convert to an average cost of time per vehicle, the cost of time per person is multiplied by the vehicle occupancy factor of 1.25 persons per vehicle. This gives an average cost of time per vehicle of $20.01.

Table 47. Factors used to convert sec/veh delay to hr/intersection delay for a year.

Traffic Period	Number of Hours in Weekday	Number of Hours in Weekend	Hours per Year[a]	Hourly Percent of ADT during Typical Weekday[b]	Hourly Percent of ADT during Typical Weekend[c]	Typical Hourly Volume If AADT Is 1000 veh/day
Weekday AM Peak Hour	2	0	522	10.0	0	100
Weekday PM Peak Hour	2	0	522	10.0	0	100
Weekday Off-Peak & Weekend Off-Peak and Peak	5	9	2243	6.1	6.1	61
Evening	7	7	2555	2.8	2.8	28
Night	8	8	2920	1.8	1.8	18
Total	24	24	8762			

[a] Assume 52.1 weeks/year with 5 days being weekdays and 2 days being weekend days.
[b] Hourly percent of traffic for given traffic period on a typical weekday
[c] Hourly percent of traffic for given traffic period on a typical weekend

Table 48. National congestion constants used in the *2009 Urban Mobility Report* (*78*). [a]

Constant	Value
Vehicle Occupancy	1.25 persons per vehicle
Working Days	250 days per year
Percent of Daily Travel in Peak Periods (6 to 10 AM and 3 to 7 PM)	50 percent
Average Cost of Time (2007)	$15.47 per person hour [b]
Commercial Vehicle Operating Cost (2007)	$102.12 per vehicle hour [b, c]

[a] Source: *2009 Urban Mobility Report* methodology, http://tti.tamu.edu/documents/mobility_report_2009_wappx.pdf
[b] Adjusted annually using the Consumer Price Index
[c] Adjusted periodically using industry cost and logistics data

CRASHES

Crash Prediction

The predicted average crash frequency for an intersection can be determined from equations in the *Highway Safety Manual* (HSM) (*77*). These equations, called safety performance functions, are regression models for estimating the predicted average crash frequency of individual roadway segments or intersections for a set of specific base conditions. As discussed in the HSM, each SPF in the predictive method was developed with observed crash data for a set of similar sites. The SPFs, like all regression models, estimate the value of a dependent variable as a function of a set of independent variables. In the SPFs developed for the HSM, the dependent variable estimated is the predicted average crash frequency for a roadway segment or intersection under base conditions, and the independent variables are the AADTs of the roadway segment or intersection legs (and, for roadway segments, the length of the roadway segment).

The SPFs applicable to the rural conditions in this study are listed in Table 49. Table 50 shows the equations for urban and suburban intersections on arterials used in this evaluation. Table 51 list the definitions for the variables listed in Table 50. Table 52 lists the acceptable ranges for average annual daily traffic for each equation. These ADT ranges were not exceeded in the evaluations.

Table 49. Safety performance functions for rural highways for total crashes.

Number of Lanes	**Number of Legs**	**Equation**	
Two	Three	$N_{spf\ 2\ ln,\ 3st} = \exp[-9.86 + 0.79 \times \ln(AADT_{maj}) + 0.49 \times \ln(AADT_{min})]$	(24)
Two	Four	$N_{spf\ 2\ ln,\ 4st} = \exp[-8.56 + 0.60 \times \ln(AADT_{maj}) + 0.61 \times \ln(AADT_{min})]$	(25)
Four	Three	$N_{spf\ 4\ ln,\ 3st} = \exp[-12.526 + 1.204 \times \ln(AADT_{maj}) + 0.236 \times \ln(AADT_{min})]$	(26)
Four	Four	$N_{spf\ 4\ ln,\ 4st} = \exp[-10.008 + 0.848 \times \ln(AADT_{maj}) + 0.448 \times \ln(AADT_{min})]$	(27)

Where:

$N_{spf\ 2\ ln,\ 3st}$ = estimate of intersection-related predicted average crash frequency for base conditions for a rural two-lane highway with three-leg stop-controlled intersections,

$N_{spf\ 2\ ln,\ 4st}$ = estimate of intersection-related predicted average crash frequency for base conditions for a rural two-lane highway with four-leg stop-controlled intersections,

$N_{spf\ 4\ ln,\ 3st}$ = estimate of intersection-related predicted average total crash frequency for base conditions for a rural four-lane highway with three-leg stop-controlled intersections,

$N_{spf\ 4\ ln,\ 4st}$ = estimate of intersection-related predicted average total crash frequency for base conditions for a rural four-lane highway with four-leg stop-controlled intersections,

$AADT_{maj}$ = AADT (vehicles per day) on the major road, and

$AADT_{min}$ = AADT (vehicles per day) on the minor road.

Table 50. Safety performance functions for urban and suburban arterials for total crashes.

No. of Legs	Crash Type	Equation	
Three	Multiple	$N_{spf\ U/S\text{-}MV,\ 3st} = \exp[-13.36 + 1.11 \times \ln(AADT_{maj}) + 0.41 \times \ln(AADT_{min})]$	(28)
Four	Multiple	$N_{spf\ U/S\text{-}MV,\ 4st} = \exp[-8.90 + 0.82 \times \ln(AADT_{maj}) + 0.25 \times \ln(AADT_{min})]$	(29)
Three	Single	$N_{spf\ U/S\text{-}SV,\ 3st} = \exp[-6.81 + 0.16 \times \ln(AADT_{maj}) + 0.51 \times \ln(AADT_{min})]$	(30)
Four	Single	$N_{spf\ U/S\text{-}SV,\ 4st} = \exp[-5.33 + 0.33 \times \ln(AADT_{maj}) + 0.12 \times \ln(AADT_{min})]$	(31)
Before Installation of Left-Turn Lane			
Three	Multiple and Single	$N_{spf\ U/S,\ 3st,\ M\&S,\ bef} = (N_{spf\ U/S\text{-}MV,\ 3st} + N_{spf\ U/S\text{-}SV,\ 3st})$	(32)
Four	Multiple and Single	$N_{spf\ U/S,\ 4st,\ M\&S,\ bef} = (N_{spf\ U/S\text{-}MV,\ 4st} + N_{spf\ U/S\text{-}SV,\ 4st})$	(33)
Three	Ped	$N_{spf\ U/S\text{-}Ped,\ 3st,\ bef} = 0.021 \times (N_{spf\ U/S,\ 3st,\ M\&S,\ bef})$	(34)
Four	Ped	$N_{spf\ U/S\text{-}Ped,\ 4st,\ bef} = 0.022 \times (N_{spf\ U/S,\ 4st,\ M\&S,\ bef})$	(35)
Three	Bike	$N_{spf\ U/S\text{-}Bike,\ 3st,\ bef} = 0.016 \times (N_{spf\ U/S,\ 3st,\ M\&S,\ bef})$	(36)
Four	Bike	$N_{spf\ U/S\text{-}Bike,\ 4st,\ bef} = 0.018 \times (N_{spf\ U/S,\ 4st,\ M\&S,\ bef})$	(37)
Three	All	$N_{spf\ U/S,\ 3st,\ bef} = N_{spf\ U/S,\ 3st,\ M\&S,\ bef} + N_{spf\ U/S\text{-}Ped,\ 3st,\ bef} + N_{spf\ U/S\text{-}Bike,\ 3st,\ bef}$	(38)
Four	All	$N_{spf\ U/S,\ 4st,\ bef} = N_{spf\ U/S,\ 4st,\ M\&S,\ bef} + N_{spf\ U/S\text{-}Ped,\ 4st,\ bef} + N_{spf\ U/S\text{-}Bike,\ 4st,\ bef}$	(39)
After Installation of Left-Turn Lane			
Three	Multiple and Single	$N_{spf\ U/S,\ 3st,\ M\&S,\ aft} = (N_{spf\ U/S\text{-}MV,\ 3st} + N_{spf\ U/S\text{-}SV,\ 3st}) \times AMF_{LTL}$	(40)
Four	Multiple and Single	$N_{spf\ U/S,\ 4st,\ M\&S,\ aft} = (N_{spf\ U/S\text{-}MV,\ 4st} + N_{spf\ U/S\text{-}SV,\ 4st}) \times AMF_{LTL}$	(41)
Three	Ped	$N_{spf\ U/S\text{-}Ped,\ 3st,\ aft} = 0.021 \times (N_{spf\ U/S,\ 3st,\ M\&S,\ aft})$	(42)
Four	Ped	$N_{spf\ U/S\text{-}Ped,\ 4st,\ aft} = 0.022 \times (N_{spf\ U/S,\ 4st,\ M\&S,\ aft})$	(43)
Three	Bike	$N_{spf\ U/S\text{-}Bike,\ 3st,\ aft} = 0.016 \times (N_{spf\ U/S,\ 3st,\ M\&S,\ aft})$	(44)
Four	Bike	$N_{spf\ U/S\text{-}Bike,\ 4st,\ aft} = 0.018 \times (N_{spf\ U/S,\ 4st,\ M\&S,\ aft})$	(45)
Three	All	$N_{spf\ U/S,\ 3st,\ aft} = N_{spf\ U/S,\ 3st,\ M\&S,\ aft} + N_{spf\ U/S\text{-}Ped,\ 3st,\ aft} + N_{spf\ U/S\text{-}Bike,\ 3st,\ aft}$	(46)
Four	All	$N_{spf\ U/S,\ 4st,\ aft} = N_{spf\ U/S,\ 4st,\ M\&S,\ aft} + N_{spf\ U/S\text{-}Ped,\ 4st,\ aft} + N_{spf\ U/S\text{-}Bike,\ 4st,\ aft}$	(47)
Crashes That Did Not Occur due to Left-Turn Lane			
Three	"Savings"	$N_{U/S,\ 3st,\ pred\text{-}saved} = N_{spf\ U/S,\ 3st,\ bef} - N_{spf\ U/S,\ 3st,\ aft}$	(48)
Four	"Savings"	$N_{U/S,\ 4st,\ pred\text{-}saved} = N_{spf\ U/S,\ 4st,\ bef} - N_{spf\ U/S,\ 4st,\ aft}$	(49)
Variable descriptions are in Table 51.			

Table 51. Definitions for variables in Table 50.

$N_{spf\ U/S\text{-}MV,\ 3st}$ = estimate of multiple-vehicle predicted average crash frequency for base conditions for urban/suburban arterial with three-leg stop-controlled intersections.

$N_{spf\ U/S\text{-}MV,\ 4st}$ = estimate of multiple-vehicle predicted average crash frequency for base conditions for urban/suburban arterial with four-leg stop-controlled intersections.

$N_{spf\ U/S\text{-}SV,\ 3st}$ = estimate of single-vehicle predicted average crash frequency for base conditions for urban/suburban arterial with three-leg stop-controlled intersections.

$N_{spf\ U/S\text{-}SV,\ 4st}$ = estimate of single-vehicle predicted average crash frequency for base conditions for urban/suburban arterial with four-leg stop-controlled intersections.

$N_{spf\ U/S,\ 3st,\ M\&S,\ bef}$ = estimate of multiple- and single-vehicle predicted average crash frequency for urban/suburban arterial with three-leg stop-controlled intersections before left-turn lane is installed.

$N_{spf\ U/S,\ 4st,\ M\&S,\ bef}$ = estimate of multiple- and single-vehicle predicted average crash frequency for urban/suburban arterial with four-leg stop-controlled intersections before left-turn lane is installed.

$N_{spf\ U/S\text{-}Ped,\ 3st,\ bef}$ = estimate of pedestrian predicted average crash frequency for urban/suburban arterial with three-leg stop-controlled intersections before left-turn lane is installed.

$N_{spf\ U/S\text{-}Ped,\ 4st,\ bef}$ = estimate of pedestrian predicted average crash frequency for urban/suburban arterial with four-leg stop-controlled intersections before left-turn lane is installed.

$N_{spf\ U/S\text{-}Bike,\ 3st,\ bef}$ = estimate of bicycle predicted average crash frequency for urban/suburban arterial with three-leg stop-controlled intersections before left-turn lane is installed.

$N_{spf\ U/S\text{-}Bike,\ 4st,\ bef}$ = estimate of bicycle predicted average crash frequency for urban/suburban arterial with four-leg stop-controlled intersections before left-turn lane is installed.

$N_{spf\ U/S,\ 3st,\ bef}$ = estimate of predicted average crash frequency for urban/suburban arterial with three-leg stop-controlled intersections before left-turn lane is installed.

$N_{spf\ U/S,\ 4st,\ bef}$ = estimate of predicted average crash frequency for urban/suburban arterial with four-leg stop-controlled intersections before left-turn lane is installed.

$N_{spf\ U/S,\ 3st,\ M\&S,\ aft}$ = estimate of multiple- and single-vehicle predicted average crash frequency for urban/suburban arterial with three-leg stop-controlled intersections after left-turn lane is installed.

$N_{spf\ U/S,\ 4st,\ M\&S,\ aft}$ = estimate of multiple- and single-vehicle predicted average crash frequency for urban/suburban arterial with four-leg stop-controlled intersections after left-turn lane is installed.

$N_{spf\ U/S\text{-}Ped,\ 3st,\ aft}$ = estimate of pedestrian predicted average crash frequency for urban/suburban arterial with three-leg stop-controlled intersections after left-turn lane is installed.

$N_{spf\ U/S\text{-}Ped,\ 4st,\ aft}$ = estimate of pedestrian predicted average crash frequency for urban/suburban arterial with four-leg stop-controlled intersections after left-turn lane is installed.

$N_{spf\ U/S\text{-}Bike,\ 3st,\ aft}$ = estimate of bicycle predicted average crash frequency for urban/suburban arterial with three-leg stop-controlled intersections after left-turn lane is installed.

$N_{spf\ U/S\text{-}Bike,\ 4st,\ aft}$ = estimate of bicycle predicted average crash frequency for urban/suburban arterial with four-leg stop-controlled intersections after left-turn lane is installed.

$N_{spf\ U/S,\ 3st,\ aft}$ = estimate of predicted average crash frequency for urban/suburban arterial with three-leg stop-controlled intersections after left-turn lane is installed.

$N_{spf\ U/S,\ 4st,\ aft}$ = estimate of predicted average crash frequency for urban/suburban arterial with four-leg stop-controlled intersections after left-turn lane is installed.

AMF_{LTL} = accident modification factor for left-turn lane—use 0.67 for three-leg or 0.73 for four-leg intersection (treatment on one approach only).

$N_{U/S,\ 3st,\ pred\text{-}saved}$ = estimate of predicted crash frequency for urban/suburban arterial with three-leg stop-controlled intersections that did *not* occur because of the installation of a left-turn lane.

$N_{U/S,\ 4st,\ pred\text{-}saved}$ = estimate of predicted crash frequency for urban/suburban arterial with four-leg stop-controlled intersections that did *not* occur because of the installation of a left-turn lane.

$AADT_{maj}$ = AADT (vehicles per day) on the major road.

$AADT_{min}$ = AADT (vehicles per day) on the minor road.

Table 52. Minimum and maximum AADT for *Highway Safety Manual* equations.

Intersection Characteristics	Major Approach Minimum to Maximum AADT	Minor Approach Minimum to Maximum AADT
Rural Two-Lane Highway with Three-Leg Stop-Controlled Intersections	0 to 19,500 veh/day	0 to 4,300 veh/day
Rural Two-Lane Highway with Four-Leg Stop-Controlled Intersections	0 to 14,700 veh/day	0 to 3,500 veh/day
Rural Four-Lane Highway with Three-Leg Stop-Controlled Intersections	0 to 78,300 veh/day	0 to 23,000 veh/day
Rural Four-Lane Highway with Four-Leg Stop-Controlled Intersections	0 to 78,300 veh/day	0 to 7,400 veh/day
Urban and Suburban Arterial Intersections with Three-Leg Stop-Controlled Intersections	0 to 45,700 veh/day	0 to 9,300 veh/day
Urban and Suburban Arterial Intersections with Four-Leg Stop-Controlled Intersections	0 to 46,800 veh/day	0 to 5,900 veh/day

For rural conditions, different SPFs are provided for two-lane and four-lane highways and for three- and four-leg intersections. For urban and suburban arterials, prediction equations are provided for three-leg and four-leg intersections with either stop control on the minor-road approaches (used in this study) or with signal control (not used in this study). Separate urban and suburban prediction equations are not provided based on the number of lanes on the major-road approach. The type of roadway segments included in the development of the SPFs and adjustment factors for urban and suburban arterials include two-lane undivided highways, three-lane arterials with center TWLTL, four-lane undivided highways, four-lane divided highways, and five-lane arterials including a center TWLTL.

The predicted average crash frequency for base conditions is adjusted using accident modification factors and a calibration factor to adjust for a particular geographical area (not used in this evaluation).

An illustration of the predicted average crashes frequency is shown in Figure 25 for the different conditions considered in this evaluation. The graph shows the predicted crashes for a range of major-road volumes when the minor-road ADT is 2000 veh/day. The predicted number of crashes for intersections on rural four-lane highways and rural two-lane four-leg intersections is higher than the crash prediction for urban and suburban arterials. The crash prediction in this illustration for rural four-lane three-leg intersections is similar to urban and suburban three-leg intersections for a given major-road average daily traffic.

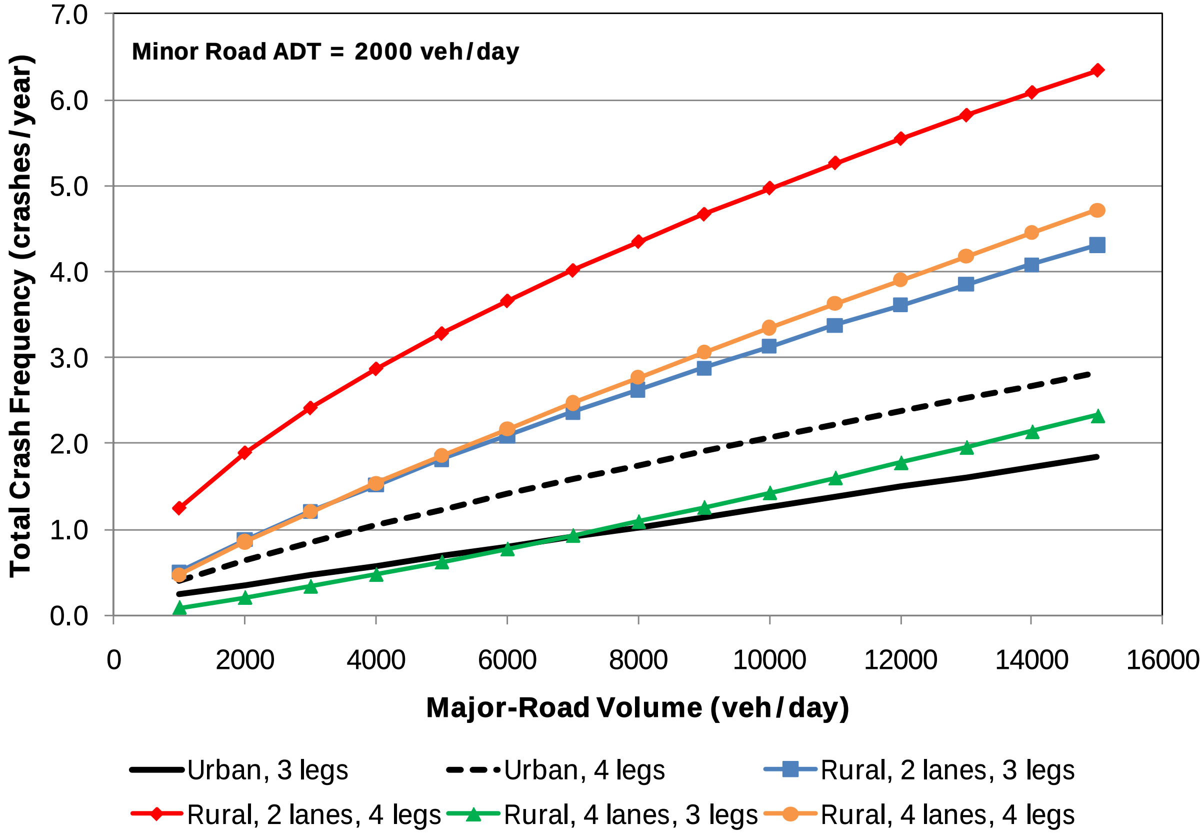


Figure 25. Illustration of predicted crash frequency using *Highway Safety Manual* equations.

Accident Modification Factor

The accident modification factor for left-turn lanes is available from the *Highway Safety Manual* (*77*). For this evaluation, the assumption was that the intersections had a stop sign on the minor approaches and that only one of the major-road approaches would be treated with a left-turn lane.

The AMFs for both the rural two-lane and four-lane highway scenarios are:

- 0.56 for a three-leg intersection and
- 0.72 for a four-leg intersection.

The AMFs for the urban and suburban scenarios are:

- 0.67 for a three-leg intersection and
- 0.73 for a four-leg intersection.

Comparison of Crash Prediction and Total Crashes at Selected Field Study Sites

Because of the newness of the *Highway Safety Manual* and the crash prediction approach, questions have been asked regarding the accuracy of the predictions. A detailed comparison between actual crashes for a given set of conditions and the prediction procedure is beyond the scope of this study. A general comparison, however, was developed for a subset of the field studies. Figure 26 illustrates the predicted average total crashes and actual total crashes (average based on 5 years of data) for a selection of the field study sites where crash data were available to the research team. While there are examples of overpredictions and underpredictions, overall the trend is as expected. Those sites with a higher number of actual crashes are also those sites with the higher predictions.

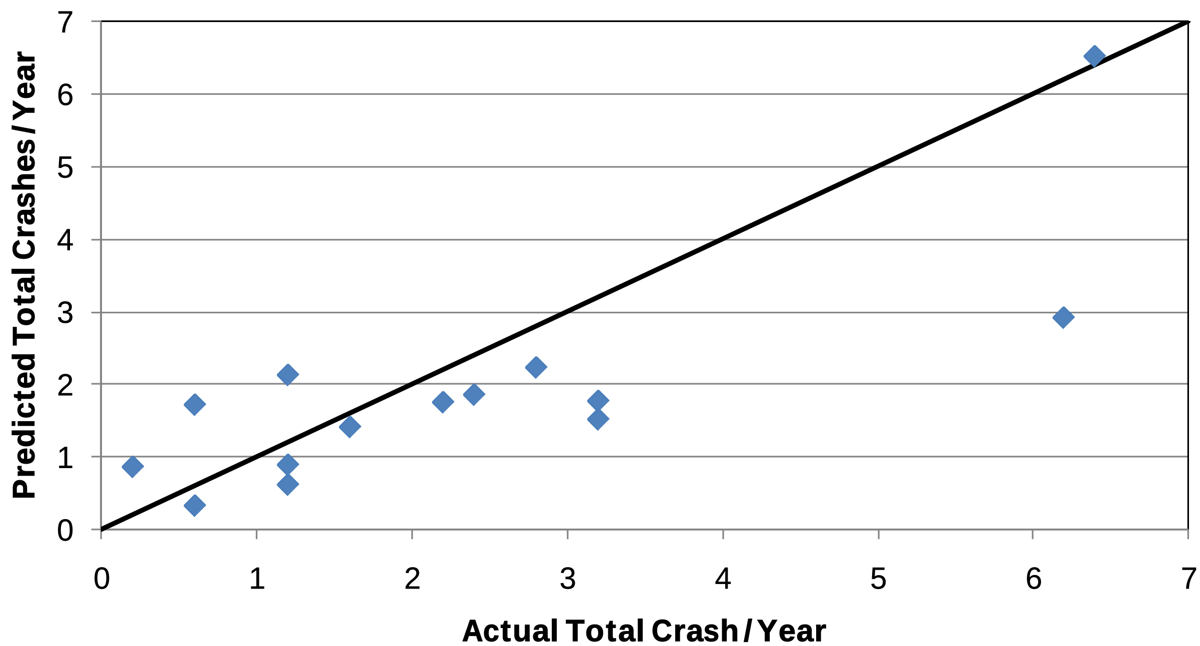


Figure 26. Comparison between actual crashes and predicted crashes for a sample of field study sites.

Volumes Used in Crash Prediction for Benefit-Cost Evaluations

Assumptions used to determine the traffic volumes used in the evaluation include the following:

- Traffic volumes in both directions of travel on the major road are the same.
- Traffic volume is evenly distributed across the lanes.
- The minor-road volume is two times the volume of left-turn volume for three-leg intersections.
- The minor-road volume is four times the volume of left-turn volume for four-leg intersections.
- At four-leg intersections, only one major-road approach has a left-turn lane. There is no crossing traffic between the minor-road legs.

2009 Value of a Statistical Life by Crash Severity

In 2008, a memo was released by the U.S. Department of Transportation regarding the treatment of the economic value of a statistical life in developmental analyses (*79*). The memo “raises to $5.8 million the value of a statistical life to be used by analysts in the Department of Transportation when assessing the benefit of preventing fatalities.”

This section describes the methodology researchers used to develop an estimate of the value of a statistical life (VSL) in 2009 dollars by crash severity. Researchers used the methodology documented in Council et al. (*80*) and subsequently implemented in the *Highway Safety Manual*. There are seven steps in the methodology that follows.

Step 1: Estimate the 2008 VSL by Crash Severity and Compute Factors for Subsequent Steps

A February 5, 2008, U.S. Department of Transportation (U.S. DOT) memorandum indicates:

> …the economic value of preventing a human fatality is $5.8 million…. In addition, we will, for the first time, require supplementary analyses at values for a statistical life higher and lower than $5.8 million. Specifically, analysts will prepare estimates based on assumptions of $3.2 million and $8.4 million for the value associated with each life saved (*79*).

With this guidance, researchers used $5.8 million as the "mid-range" comprehensive societal cost in 2008 dollars (the date of the U.S. DOT memorandum). Researchers used a "low" value of $3.2 million and a "high" value of $8.4 million for the subsequent computations. The *Highway Safety Manual* (*77*) provides estimates of the human capital costs and comprehensive societal costs by crash severity in 2001 dollars. These values are shown in Table 53. The fourth and fifth columns of Table 53 show factors used in subsequent computations.

Table 53. Human capital cost and comprehensive societal cost from the *Highway Safety Manual* with factors for subsequent analyses.[a]

Crash Severity	Human Capital Cost [b] (2001 Dollars)	Comprehensive Societal Cost [b] (2001 Dollars)	Comprehensive Societal Cost for a Given Crash Severity Relative to a Fatality[c]	Comprehensive Societal Cost Relative to Human Capital Cost[d]
Fatality (K)	$1,245,600	$4,008,900	1.0000	3.2184
Disabling Injury (A)	$111,400	$216,000	0.0539	1.9390
Evident Injury (B)	$41,900	$79,000	0.0197	1.8854
Possible Injury (C)	$28,400	$44,900	0.0112	1.5810
PDO	$6,400	$7,400	0.0018	1.1563

[a]2001 dollars as adapted from the *Highway Safety Manual* (*77*)
[b]From Table 4A-1, "Crash Costs Estimates by Crash Severity," from the *Highway Safety Manual* (*77*)
[c]For example, for a disabling injury (A), computed as $216,000 divided by $4,008,900
[d]For example, for a disabling injury (A), computed as $216,000 divided by $111,400

Step 2: Estimate 2008 Comprehensive Societal Cost from U.S. DOT Memorandum Using Factors Developed in Step 1

Table 54 shows the low, mid-range, and high comprehensive societal cost of a fatality based on the U.S. DOT memorandum identified in Step 1. Costs by crash severity are computed with the "Comprehensive Societal Cost for a Given Crash Severity Relative to a Fatality" values shown in

Table 53. For example, the low disabling injury (A) comprehensive societal cost is computed as $3,200,000 multiplied by 0.0539 (from Table 53), which gives $172,400.

Table 54. Low, mid-range, and high comprehensive societal cost estimates (2008 dollars).

Crash Severity	Comprehensive Societal Cost (Low)	Comprehensive Societal Cost (Mid-range)	Comprehensive Societal Cost (High)
Fatality (K)	$3,200,000	$5,800,000	$8,400,000
Disabling Injury (A)	$172,400	$312,500	$452,600
Evident Injury (B)	$63,100	$114,300	$165,500
Possible Injury (C)	$35,800	$65,000	$94,100
PDO	$5,900	$10,700	$15,500
Note: Values are rounded after spreadsheet calculations.			

Step 3: Estimate 2008 Human Capital Costs Using Factors Developed in Step 1

Researchers estimated the human capital costs by crash severity using the factors in the fifth column of Table 53. The factors in Table 53 (comprehensive societal cost relative to human capital cost factors) are the ratio of the comprehensive societal cost to the human capital cost. The low fatality (K) human capital cost is computed as $3,200,000 (from Table 54) divided by 3.2184 (from Table 53), which gives $994,300. The other values in Table 55 are computed in a similar manner.

Table 55. Low, mid-range, and high human capital cost estimates (2008 dollars).

Crash Severity	Human Capital Cost (Low)	Human Capital Cost (Mid-range)	Human Capital Cost (High)
Fatality (K)	$994,300	$1,802,100	$2,610,000
Disabling Injury (A)	$88,900	$161,200	$233,400
Evident Injury (B)	$33,400	$60,600	$87,800
Possible Injury (C)	$22,700	$41,100	$59,500
PDO	$5,100	$9,300	$13,400
Note: Values are rounded after spreadsheet calculations.			

Step 4: Estimate Cost Difference between Comprehensive Societal Cost Estimates and Human Capital Cost by Crash Severity

In this step, researchers computed the difference between the comprehensive societal cost estimates in Table 54 and the human capital cost estimates in Table 55. These values are shown in Table 56. For example the mid-range cost difference of $1,400 in Table 56 for PDO was determined as $10,700 (in Table 54) minus $9,300 (in Table 55).

Table 56. Cost difference between comprehensive societal cost estimates and human capital cost estimates (2008 dollars).

Crash Severity	Cost Difference (Low)	Cost Difference (Mid-range)	Cost Difference (High)
Fatality (K)	$2,205,700	$3,997,900	$5,790,000
Disabling Injury (A)	$83,500	$151,300	$219,200
Evident Injury (B)	$29,600	$53,700	$77,700
Possible Injury (C)	$13,200	$23,900	$34,600
PDO	$800	$1,400	$2,100
Note: Values are rounded after spreadsheet calculations.			

Step 5: Estimate the 2009 Human Cost by Crash Severity

Researchers adjusted the 2008 human capital cost estimates using the CPI. From the U.S. Bureau of Labor Statistics, the CPI for 2008 is 215.303, and the CPI for 2009 is 214.537 (*81*). The ratio of the 2009 CPI to the 2008 CPI value (214.537 ÷ 215.303) is 0.996. Researchers multiplied the values in Table 55 by 0.996 to obtain the 2009 CPI-adjusted human capital cost estimates in Table 57.

Table 57. 2009 CPI-adjusted human capital cost estimates (2009 dollars).

Crash Severity	Human Capital Cost (Low)	Human Capital Cost (Mid-range)	Human Capital Cost (High)
Fatality (K)	$990,700	$1,795,700	$2,600,700
Disabling Injury (A)	$88,600	$160,600	$232,600
Evident Injury (B)	$33,300	$60,400	$87,500
Possible Injury (C)	$22,600	$40,900	$59,300
PDO	$5,100	$9,200	$13,400
Note: Values are rounded after spreadsheet calculations.			

Step 6: Estimate the 2009 Cost Difference between Comprehensive Societal Cost and Human Capital Cost by Crash Severity

Researchers adjusted the 2008 cost differences in Table 56 to 2009 dollars using the employment cost index (ECI). Researchers obtained quarterly ECI values from the U.S. Bureau of Labor Statistics (*82*). The quarterly values were averaged to obtain an annual value for 2008 (108.65) and 2009 (110.5). The ratio of the 2009 value to the 2008 value (110.5 ÷ 108.65) is 1.02. Researchers multiplied the values in Table 56 by 1.02 to obtain the 2009 ECI-adjusted cost differences by crash severity as shown in Table 58.

Table 58. 2009 ECI-adjusted cost difference between comprehensive societal cost estimates and human capital cost estimates (2009 dollars).

Crash Severity	Cost Difference (Low)	Cost Difference (Mid-range)	Cost Difference (High)
Fatality (K)	$2,243,300	$4,066,000	$5,888,600
Disabling Injury (A)	$84,900	$153,900	$222,900
Evident Injury (B)	$30,100	$54,600	$79,100
Possible Injury (C)	$13,400	$24,300	$35,200
PDO	$800	$1,500	$2,100
Note: Values are rounded after spreadsheet calculations.			

Step 7: Estimate 2009 Comprehensive Societal Cost Estimates

Researchers estimated the 2009 total comprehensive societal costs by summing the 2009 human capital cost estimates (Table 57) and the 2009 cost differences (Table 58). This sum is shown in Table 59.

Table 59. 2009 Comprehensive societal cost estimates (2009 dollars).

Crash Severity	Comprehensive Societal Cost (Low)	Comprehensive Societal Cost (Mid-range)	Comprehensive Societal Cost (High)
Fatality (K)	$3,234,000	$5,861,700	$8,489,300
Disabling Injury (A)	$173,500	$314,500	$455,500
Evident Injury (B)	$63,400	$115,000	$166,500
Possible Injury (C)	$36,000	$65,200	$94,500
PDO	$5,900	$10,700	$15,500
Note: Values are rounded after spreadsheet calculations.			

Typical Crash Cost for Three-Leg and Four-Leg Intersections

The cost per crash at a three-leg or a four-leg intersection requires knowing the distribution of crash severity for the different intersection configurations. Table 10-5 in the *Highway Safety Manual* (*77*) provides the default proportions for crash severity levels for three-leg and four-leg stop-controlled rural intersections (reproduced as Table 60 in this report).

Also needed is the conversion of the cost per person to a cost per crash. The number of individuals killed or injured in a crash is not readily available. A study on red-light running at signalized intersections in Texas identified the number of annual crashes and the number of annual injuries by severity level (*83*). From those values, the number of injuries or fatalities can be calculated as shown in Table 61.

Table 60. Default distribution of crash severity level at rural two-lane two-way intersections from the *Highway Safety Manual* (*77*).

Crash Severity Level	Percentage of Total Crashes		
	Three-Leg Stop-Controlled Intersections	Four-Leg Stop-Controlled Intersections	Four-Leg Signalized Intersections
Fatality	1.7	1.8	0.9
Incapacitating Injury	4.0	4.3	2.1
Nonincapacitating Injury	16.6	16.2	10.5
Possible Injury	19.2	20.8	20.5
Property Damage Only	58.5	56.9	66.0
Total	100.0	100.0	100.0

Table 61. Injuries per crash for red-light-running crashes (*83*).

Severity	Annual Crashes	Annual Injuries or Deaths	Injuries or Deaths/Crash
K	121	133	1.10 deaths/crash
A	1,439	2,047	1.42 injuries/crash
B	5,493	8,987	1.64 injuries/crash
C	11,798	24,802	2.10 injuries/crash
PDO	18,851	0	0.00 injuries/crash

A current Texas Department of Transportation (TxDOT) study is examining crashes at rural intersections. Data available for 595 rural intersections provided the distributions shown in Table 62. For the 1198 crashes, the number of injured persons per crash ranged between 1.22 and 2.30. The fatal crashes had 1.09 deaths per crash at the four-leg intersections and 1.46 deaths per crash at the three-leg intersections. Reflecting the multiple conflict points at an intersection, the average number of vehicles involved at a crash ranged between 1.48 and 2.36 veh/crash. Table 63 and Table 64 show the calculations to determine typical crash cost using the ranges for comprehensive societal cost for three-leg and four-leg rural highway intersections, respectively. The calculations to determine typical costs for urban suburban arterials are shown in Table 65.

The *Highway Safety Manual* also provides crash cost estimates by crash severity. These values are listed in Table 66. The crash costs listed in Table 66 were converted to 2009 dollars, and then the typical crash costs by number of legs and rural or urban were determined (see Table 67). The values in Table 66 are costs per crash, while the previous calculations had to convert the cost for a statistical life into cost per crash. The crash cost values in the *Highway Safety Manual* are assumed to already have accounted for the number of persons typically involved in a crash along with the distribution of injuries within a crash. The HSM values, however, do not account for the higher value being placed on a statistical life. Therefore, most of the B/C calculations were performed using the values listed in Table 63, Table 64, and Table 65. A comparison is made, however, at the end of this section with the left-turn lane warrants that would result if the costs in Table 67 are assumed.

Table 62. Injuries or deaths per crash for rural intersections.[a]

Severity	Injuries or Deaths/Crash		Number of Persons/Crash		Number of Vehicles/Crash	
	Three Legs	Four Legs	Three Legs	Four Legs	Three Legs	Four Legs
K	1.46 deaths/crash 0.31 A injuries/crash 1.15 B injuries/crash 0.00 C injuries/crash 0.31 no injuries/crash 0.15 unk. injuries/crash	1.09 deaths/crash 0.55 A injuries/crash 0.55 B injuries/crash 0.36 C injuries/crash 0.36 no injuries/crash 0.36 unk. injuries/crash	3.38	3.27	1.54	2.36
A	1.17 A injuries/crash 0.29 B injuries/crash 0.12 C injuries/crash 0.38 no injuries/crash 0.06 unk. injuries/crash	1.40 A injuries/crash 0.47 B injuries/crash 0.43 C injuries/crash 1.83 no injuries/crash 0.10 unk. injuries/crash	2.01	4.23	1.48	2.00
B	1.30 B injuries/crash 0.18 C injuries/crash 0.59 no injuries/crash 0.09 unk. injuries/crash	1.42 B injuries/crash 0.48 C injuries/crash 1.08 no injuries/crash 0.08 unk. injuries/crash	2.15	3.06	1.55	1.87
C	1.22 C injuries/crash 0.89 no injuries/crash 0.13 unk. injuries/crash	1.34 C injuries/crash 1.20 no injuries/crash 0.09 unk. injuries/crash	2.24	2.64	1.53	1.82
PDO	0.00 injuries/crash	0.00 injuries/crash	2.15	2.48	1.61	1.88

[a] Findings based on 1189 crashes at 595 rural Texas intersections for the time period of 2003 to 2008

Unk. = unknown

Table 63. Typical crash cost calculations for three-leg rural intersections.

Range (Cost)	Crash Severity	Injury Severity	Cost [a, b]	Convert Cost/Person to Cost/Crash [c]	Cost per Crash	Percent of Total Crashes [d]	Extension
Mid-range ($214,000)	Fatality	K A B C	$5,861,700 $314,500 $115,000 $65,200	1.46 0.31 1.15 0.00	$8,558,082 $97,495 $132,250 $0	1.70	$149,393
	A	A B C	$314,500 $115,000 $65,200	1.17 0.29 0.12	$367,965 $33,350 $7,824	4.00	$16,366
	B	B C	$115,000 $65,200	1.30 0.18	$149,500 $11,736	16.60	$26,765
	C	C	$65,200	1.22	$79,544	19.20	$15,272
	PDO	PDO	$10,700	1.00 [e]	$10,700	58.50	$6,260
					Total (cost/crash)	**100.00**	**$214,056**
Low ($118,000)	Fatality	K A B C	$3,234,000 $173,500 $63,400 $36,000	1.46 0.31 1.15 0.00	$4,721,640 $35,785 $72,910 $0	1.70	$82,422
	A	A B C	$173,500 $63,400 $36,000	1.17 0.29 0.12	$202.995 $18,386 $4,320	4.00	$9,028
	B	B C	$63,400 $36,000	1.30 0.18	$82,420 $6,480	16.60	$14,757
	C	C	$36,000	1.22	$43,920	19.20	$8,433
	PDO	PDO	$5,900	1.00 [e]	$5,900	58.50	$3,452
					Total (cost/crash)	**100.00**	**$118,091**
High ($310,000)	Fatality	K A B C	$8,489,300 $455,500 $166,500 $94,500	1.46 0.31 1.15 0.00	$12,394,378 $141,205 $191,475 $0	1.70	$216,360
	A	A B C	$455,500 $166,500 $94,500	1.17 0.29 0.12	$532,935 $48,285 $11,340	4.00	$23,702
	B	B C	$166,500 $94,500	1.30 0.18	$216,450 $17,010	16.60	$28,754
	C	C	$94,500	1.22	$115,290	19.20	$22,136
	PDO	PDO	$15,500	1.00 [e]	$15,500	58.50	$9,068
					Total (cost/crash)	**100.00**	**$310,020**

[a] Comprehensive societal cost for fatal crash is from "Treatment of the Economic Value of a Statistical Life in Departmental Analyses," Memorandum to Secretarial Officers, Modal Administrators, available at http://ostpxweb.dot.gov/policy/reports/080205.htm.
[b] Comprehensive societal cost for crash severity A, B, C, or PDO is based on distribution determined using *Highway Safety Manual* data (see Table 53), with costs adjusted to 2009 dollars.
[c] Factors from Table 62
[d] From Table 10-5 of the *Highway Safety Manual* (*77*)
[e] No factor is needed. Assumption is that cost reflects cost per crash.

Table 64. Typical crash cost calculations for four-leg intersections.

Range (Cost)	Crash Severity	Injury Severity	Cost [a, b]	Convert Cost/Person to Cost/Crash [c]	Cost per Crash	Percent of Total Crashes [d]	Extension
Mid-range ($198,000)	Fatality	K A B C	$5,861,700 $314,500 $115,000 $65,200	1.09 0.55 0.55 0.36	$6,389,253 $172,975 $63,250 $23,472	1.80	$119,681
	A	A B C	$314,500 $115,000 $65,200	1.40 0.47 0.43	$440,300 $54,050 $28,036	4.30	$22,463
	B	B C	$115,000 $65,200	1.42 0.48	$163,300 $31,296	16.20	$31,525
	C	C	$65,200	1.34	$87,368	20.80	$18,173
	PDO	PDO	$10,700	1.00 [e]	$10,700	56.90	$6,088
					Total (cost/crash)	**100.00**	**$197,929**
Low ($109,000)	Fatality	K A B C	$3,234,000 $173,500 $63,400 $36,000	1.09 0.55 0.55 0.36	$3,525,060 $95,425 $34,870 $12,960	1.80	$66,030
	A	A B C	$173,500 $63,400 $36,000	1.40 0.47 0.43	$242,900 $29,798 $15,480	4.30	$12,392
	B	B C	$63,400 $36,000	1.42 0.48	$90,028 $17,280	16.20	$17,384
	C	C	$36,000	1.34	$48,240	20.80	$10,034
	PDO	PDO	$5,900	1.00 [e]	$5,900	56.90	$3,357
					Total (cost/crash)	**100.00**	**$109,196**
High ($287,000)	Fatality	K A B C	$8,489,300 $455,500 $166,500 $94,500	1.09 0.55 0.55 0.36	$9,253,337 $250,525 $91,575 $34,020	1.80	$173,330
	A	A B C	$455,500 $166,500 $94,500	1.40 0.47 0.43	$637,700 $78,255 $40,635	4.30	$32,522
	B	B C	$166,500 $94,500	1.42 0.48	$236,420 $45,360	16.20	$45,650
	C	C	$94,500	1.34	$126,630	20.80	$26,339
	PDO	PDO	$15,500	1.00 [e]	$15,500	56.90	$8,820
					Total (cost/crash)	**100.00**	**$286,672**

[a] Comprehensive societal cost for fatal crash is from "Treatment of the Economic Value of a Statistical Life in Departmental Analyses," Memorandum to Secretarial Officers, Modal Administrators, available at http://ostpxweb.dot.gov/policy/reports/080205.htm.
[b] Comprehensive societal cost for crash severity A, B, C, or PDO is based on distribution determined using *Highway Safety Manual* data (see Table 53), with costs adjusted to 2009 dollars.
[c] Factors from Table 62
[d] From Table 10-5 of the *Highway Safety Manual* (*77*)
[e] No factor is needed. Assumption is that cost reflects cost per crash.

Table 65. Typical crash cost calculations for urban and suburban intersections.

Range (Cost)	Crash Severity	Cost [a, b]	Convert Cost/ Person to Cost/ Crash [c]	Cost per Crash	Percent of Total Crashes for Three Leg [d]	Extension for Three Legs	Percent of Total Crashes for Four Legs [d]	Extension for Four Legs
Mid-range	Fatality	$5,861,700	1.10	$6,447,870	1.47	$95,087	1.60	$103,405
	A	$314,500	1.42	$446,590	3.47	$15,496	3.83	$17,109
	B	$115,000	1.64	$188,600	14.40	$27,158	14.43	$27,221
	C	$65,200	2.10	$136,920	16.66	$22,805	18.53	$25,374
	PDO	$10,700	1.00 [e]	$10,700	64.00	$6,848	61.60	$6,591
	Total (cost/crash)					$167,394		$179,701
	Rounded total (cost/crash)					**$167,000**		**$180,000**
Low	Fatality	$3,234,000	1.10	$3,557,400	1.47	$52,461	1.60	$57,050
	A	$173,500	1.42	$246,370	3.47	$8,549	3.83	$9,439
	B	$63,400	1.64	$103,976	14.40	$14,973	14.43	$15,007
	C	$36,000	2.10	$75,600	16.66	$12,591	18.53	$14,010
	PDO	$5,900	1.00 [e]	$5,900	64.00	$3,776	61.60	$3,634
	Total (cost/crash)					$92,350		$99,141
	Rounded total (cost/crash)					**$92,000**		**$99,000**
High	Fatality	$8,489,300	1.10	$9,338,230	1.47	$137,711	1.60	$149,758
	A	$455,500	1.42	$646,810	3.47	$22,444	3.83	$24,780
	B	$166,500	1.64	$273,060	14.40	$39,321	14.43	$39,412
	C	$94,500	2.10	$198,450	16.66	$33,053	18.53	$36,776
	PDO	$15,500	1.00 [e]	$15,500	64.00	$9,920	61.60	$9,548
	Total (cost/crash)					$242,448		$260,274
	Rounded total (cost/crash)					**$242,000**		**$260,000**

[a] Comprehensive societal cost for fatal crash is from "Treatment of the Economic Value of a Statistical Life in Departmental Analyses," Memorandum to Secretarial Officers, Modal Administrators, available at http://ostpxweb.dot.gov/policy/reports/080205.htm.
[b] Comprehensive societal cost for crash severity A, B, C, or PDO is based on distribution determined using *Highway Safety Manual* data (see Table 53), with costs adjusted to 2009 dollars.
[c] Factors from Table 61
[d] Table 52 from *Methodology to Predict the Safety Performance of Urban and Suburban Arterials* (*84*) shows PDO crashes to be 64.0 percent for multiple-vehicle crashes at three-leg stop-controlled intersections and 61.6 percent for four-leg stop-controlled intersections. The remaining 36.0 percent for three-leg and 38.4 percent for four-leg intersections were distributed between fatality, A, B, and C using similar proportions as assumed for rural highways.
[e] No factor is needed. Assumption is that cost reflects cost per crash.

Table 66. Crash cost estimates by crash severity from the *Highway Safety Manual* (*77*).

Crash Severity Level	Human Capital Crash Costs (2001 Dollars)	Comprehensive Crash Costs (2001 Dollars)
Fatality	$1,245,600	$4,008,900
Incapacitating Injury	$111,400	$216,000
Nonincapacitating Injury	$41,900	$79,000
Possible Injury	$28,400	$44,900
Property Damage Only	$6,400	$7,400

Source: references within the *Highway Safety Manual to* Council et al., *Crash Cost Estimates by Maximum Police-Reported Injury Severity within Selected Crash Geometries,* FHWA-HRT-05-051, October 2005.

Table 67. Typical crash cost by number of legs and rural or urban based on *Highway Safety Manual* crash costs.

Crash Severity Level	Comprehensive Crash Costs [a] (2009 Dollars)	Rural		Urban and Suburban	
		3 Legs	4 Legs	3 Legs	4 Legs
Crash Severity Distribution					
Fatality (K)	$5,059,425	1.7	1.8	1.5	1.6
Disabling Injury (A)	$269,348	4.0	4.3	3.5	3.8
Evident Injury (B)	$98,426	16.6	16.2	14.4	14.4
Possible Injury (C)	$55,604	19.2	20.8	16.7	18.5
PDO (O)	$9,038	58.5	56.9	64.0	61.6
Typical Crash Cost					
Typical Crash Cost per Number of Legs and Rural or Urban Based on HSM Data		$129,086	$135,305	$112,941	$121,340

[a] Comprehensive crash cost for crash severity from *Highway Safety Manual* Table 4A-1 adjusted to 2009 dollars

CONSTRUCTION COSTS

Typical construction costs for left-turn lanes were identified from several sources. Construction costs were identified in two documents. The FHWA study on safety effectiveness of turn lanes by Harwood et al. (*2*) used an average of $85,000 to be the cost associated with installing a left-turn lane, based on estimates from four of the states that participated in that study. The value of $100,000 was assumed as the cost for constructing a right-turn lane in the Potts et al. (*58*) work. The amount was selected as “a reasonable value that would be substantial to require in-depth analysis but not overly cost-prohibitive.” Based on the date of their reports, it is assumed to represent 2005 dollars. Table 68 lists the estimated costs identified in the literature along with the equivalent 2009 dollar values using the Consumer Price Index (*81*).

State department of transportation websites were also searched for information on bids for letting a left-turn lane project. Table 69 lists a compilation of recently let (2009 or 2010) projects that involved installation of a left-turn lane. Data from the following four states were obtained: Texas (*85*), Louisiana (*86*), Ohio (*87*), and Florida (*88*).

A reasonable range for the cost of constructing a left-turn lane appears to be $100,000 to $375,000 with an average value of $250,000. These values are used to represent the following range listed in the evaluation:

- $100,000 to represent minimal (min) costs,
- $250,000 to represent moderate (mod) costs, and
- $375,000 to represent maximum (max) costs within the range studied in this evaluation.

Table 68. Estimated construction cost from literature.

Study	Estimated Construction Cost for Turn Lane ($1,000)	Assumed Year of Cost	Consumer Price Index for Year of Estimate	Consumer Price Index for 2009	Construction Cost in 2009 Dollars ($1,000)
FHWA Safety of Turn Lanes (*2*)	85	2001	177.1	214.537	103
Right-Turn Lane (*58*)	100	2005	195.3	214.537	110

Table 69. Construction cost for left-turn lane projects from four states.

State	Let Date	Highway	County	Cost ($1,000)
Texas	December 2009	SH 95	Bastrop	250
Texas	December 2009	SH 95	Bastrop	120
Texas	December 2009	SH 95	Bastrop	250
Texas	April 2010	US 281	Blanco	200
Texas	April 2010	US 281	Blanco	250
Texas	April 2010	US 281	Blanco	140
Texas	April 2010	SH 29	Burnet	345
Texas	March 2010	FM 969	Travis	180
Texas	March 2010	FM 969	Travis	280
Texas	March 2010	FM 1327	Travis	320
Texas	July 2010	RM 1431	Travis	393
Texas	May 2010	US 183	Travis	400
Texas	May 2010	US 183	Travis	170
Texas	**Rounded average**			**254**
Louisiana	July 2010	LA 22	Ascension	750 [a]
Louisiana	October 2010	US 71	Bossier	375 [b]
Louisiana	August 2010	LA 428	Orleans	375 [b]
Louisiana	August 2010	LA 594	Ouachita	375 [b]
Louisiana	June 2010	US 90	St. Charles	375 [b]
Louisiana	July 2010	LA 22	Tangipahoa	175 [c]
Louisiana	November 2010	LA 10	Washington	175 [c]
Louisiana	**Rounded average**			**371**
Ohio	November 2010	SR 78	Woodsfield	155
Ohio	**Rounded average**			**155**
Florida	October 2009	333312-1-52-07		223
Florida	October 2009	333324-1-52-07		292
Florida	October 2009	333325-1-52-07		250
Florida	**Rounded average**			**255**
Rounded average for projects listed in table				**250**

[a] Represents middle of estimated range of $500,000 to $1,000,000
[b] Represents middle of estimated range of $250,000 to $500,000
[c] Represents middle of estimated range of $100,000 to $250,000

EXAMPLES OF CALCULATIONS FOR ADDING LEFT-TURN LANE AT EXISTING SITE

Adding Left-Turn Lane at Existing Site on Rural Two-Lane Highway

Annual Delay Savings

The following illustrates the calculations to determine if a left-turn lane should be considered at a rural two-lane highway site. Assume that the conditions at the site include the following:

- Two lanes,
- Rural location,
- Three legs,
- 50-mph posted speed limit,
- 450 veh/hr/ln in the peak hour (10,000 ADT on the major road),
- 100 veh/hr turning left in the peak hour, and
- 2000 ADT on the minor road.

The amount of expected delay reduction that would be attributed to the left-turn lane during the peak hour is 0.701 sec/veh. This value was determined using the regression coefficients listed in Table 45; however, it could also be determined from Figure 23. The delay reduction represents the per-vehicle delay savings. To determine the savings for all the vehicles at the intersection for that hour, the delay reduction value needs to be multiplied by the number of vehicles at the intersection for that hour, in this example 1000 veh (450 through and right-turning vehicles per lane on each approach along with 100 left-turning veh). Therefore 701 sec of delay was reduced during the AM peak hour. With 522 hours in that traffic period, the amount of delay savings for the year is 102 hours. The amount of delay savings per traffic period is shown in Table 70. For this site, the annual delay saving is $4,214 assuming a per-hour vehicle cost of $20.01.

Annual Crash Savings

The determination of the savings attributed to crashes begins with calculating the predicted number of crashes for the intersection. The equation for a three-leg intersection is:

$$N_{spf\ 2\ ln,\ 3st} = \exp[-9.86 + 0.79 \times \ln(AADT_{maj}) + 0.49 \times \ln(AADT_{min})] \qquad (50)$$

$$N_{spf\ 2\ ln,\ 3st} = \exp[-9.86 + 0.79 \times \ln(10{,}000) + 0.49 \times \ln(2000)]$$

$$N_{spf\ 2\ ln,\ 3st} = 3.13 \text{ crashes/year}$$

Table 70. Calculation of annual delay savings for rural two-lane highway example.

Traffic Period	Hourly Factor	Hours/ Year	V_{TR} (veh/ hr)	V_{LT} (veh/ hr)	DR (sec/ veh)	DR (sec) in the Hour	DR (sec/Year)	DR (hr/Year)
AM Peak	10	522	450	100	0.701	701.1	36,5987	101.66
PM Peak	10	522	450	100	0.701	701.1	36,5987	101.66
Off Peak/ Weekend Peak	6.1	2243	274.5	61	0.010	6.1	13,682	3.80
Evening	2.8	2555	126	28	0.010	2.8	7,154	1.99
Night	1.8	2920	81	18	0.010	1.8	5,256	1.46
							Total	210.57
							Dollars per Vehicle Hour Cost	$20.01
							Annual Delay Savings	$4,213.59

V_{TR} = Through and right-turn volume on major approach
V_{LT} = Left-turn volume
DR = Delay reduction

The accident modification factor for adding a left-turn lane on one approach for a three-leg intersection is 0.56. The reduction in number of crashes at the intersection can be determined as follows:

$$N_{spf\ 2\ ln,\ 3st} - w/LTL = N_{spf\ 2\ ln,\ 3st}\,(AMF_{LTL}) = 3.13\ (0.56) = 1.75 \text{ crashes/year} \quad (51)$$

$$N_{spf\ 2\ ln,\ 3st} - \text{savings due LTL} = N_{spf\ 2\ ln,\ 3st} - N_{spf\ 2\ ln,\ 3st} - w/LTL = 3.13 - 1.75 = 1.38 \text{ crashes/year} \quad (52)$$

The costs per crash at three-leg intersections are:

- $214,000 (average),
- $118,000 (low range), and
- $310,000 (high range).

Multiplying the cost per crash by the number of crashes that would not occur due to the presence of the left-turn lane results in the following annual crash savings:

- $294,596 (average),
- $162,441 (low range), and
- $426,752 (high range).

Present Worth of Annual Savings and Benefit-Cost Ratio

The delay and crash savings above represent the expected value per year. Assuming a 20-year design life and a 4 percent return, those annual savings need to be converted to a present worth so that the expected construction cost of the left-turn lane can be considered. The construction cost was estimated to be $250,000.

The B/C ratio when using the mid-range societal cost is:

$$\frac{B}{C} = \frac{13.59 \times (\$4{,}214 + \$294{,}596)}{\$250{,}000} = 16.2 \qquad (53)$$

The B/C ratio of 16.2 indicates that the installation of a left-turn lane should be warranted for these conditions.

Adding Left-Turn Lane on Existing Rural Four-Lane Highway

Annual Delay Savings

The following illustrates the calculations to determine if a left-turn lane should be considered at a rural four-lane highway site. Assume that the conditions at the site include the following:

- Four lanes on the major road,
- Rural highway,
- Four legs,
- 30-mph posted speed limit,
- 350 veh/hr/ln in the peak hour (16,000 ADT on the major road),
- 100 veh/hr turning left in the peak hour, and
- 4,000 ADT on the minor road.

The amount of expected delay reduction that would be attributed to the left-turn lane during the peak hour is 0.28 sec/veh. This value was determined using the regression coefficients; however, it could also be determined from Figure 23. The amount of delay savings per traffic period is shown in Table 71. For this site, the annual delay saving is $1,514 assuming a per-hour vehicle cost of $20.01.

Annual Crash Savings

The determination of the savings attributed to crashes begins with calculating the predicted number of crashes for the intersection. The equation for a three-leg intersection is:

$$N_{spf\,4ln,\,4st} = \exp[-10.01 + 0.85 \times \ln(AADT_{maj}) + 0.45 \times \ln(AADT_{min})] \qquad (54)$$

$$N_{spf\,4ln,\,4st} = \exp[-10.01 + 0.85 \times \ln(16{,}000) + 0.49 \times \ln(4{,}000)]$$

$$N_{spf\,4ln,\,4st} = 6.798 \text{ crashes/year}$$

Table 71. Calculation of annual delay savings for rural four-lane highway example.

Traffic Period	Hourly Factor	Hours/ Year	V_{TR} (veh/ hr/ln)	V_{LT} (veh/ hr)	DR (sec/ veh)	DR (sec/Year)	DR (hr/Year)
AM Peak	10	522	375	100	0.28	12,5107	34.75
PM Peak	10	522	375	100	0.28	12,5107	34.75
Off Peak/ Weekend Peak	6.1	2243	229	61	0.01	11,630	3.23
Evening	2.8	2555	105	28	0.01	6,081	1.69
Night	1.8	2920	68	18	0.01	4,468	1.24
Total							75.66
Dollars per Vehicle Hour Cost							$20.01
Annual Delay Savings							$1,514
V_{TR} = Through and right-turn volume on major approach V_{LT} = Left-turn volume DR = Delay reduction							

The accident modification factor for adding a left-turn lane on one approach for a four-leg intersection is 0.72. The reduction in number of crashes at the intersection can be determined as follows:

$$N_{spf\ 4\ ln,\ 4st} - w/LTL = N_{spf\ 4\ ln,\ 4st} \times (AMF_{LTL}) = 6.798 \times (0.72) = 4.895 \text{ crashes/year} \quad (55)$$

$$N_{spf\ 4\ ln,\ 4st} - \text{savings due LTL} = N_{spf\ 4\ ln,\ 4st} - N_{spf\ 4\ ln,\ 4st\ -\ w/LTL}$$

$$= 6.798 - 4.895 = 1.904 \text{ crashes/year} \quad (56)$$

The costs per crash at three-leg intersections are:

- $198,000 (mid-range),
- $109,000 (low range), and
- $287,000 (high range).

Multiplying the cost per crash by the number of crashes that would not occur due to the presence of the left-turn lane results in the following annual crash savings:

- $376,895 (mid-range),
- $207,483 (low range), and
- $546,308 (high range).

Present Worth of Annual Savings and Benefit-Cost Ratio

The delay and crash savings above represent the expected value per year. Assuming a 20-year design life and a 4 percent return, those annual savings need to be converted to a present worth so that the expected construction cost of the left-turn lane can be considered. The construction cost was estimated to be $250,000.

The B/C ratio when using the mid-range societal cost is:

$$\frac{B}{C} = \frac{13.59 \times (\$1{,}514 + \$376{,}895)}{\$250{,}000} = 20.6 \tag{57}$$

The B/C ratio of 20.6 indicates that the installation of a left-turn lane should be warranted for these conditions.

Adding Left-Turn Lane to Existing Urban and Suburban Intersection

Annual Delay Savings

The following illustrates the calculations to determine if a left-turn lane should be considered at a site. Assume that the conditions at the site include the following:

- Four lanes on the major road,
- Urban arterial,
- Three legs,
- 40-mph posted speed limit,
- 350 veh/hr/ln in the peak hour (14,000 ADT on the major road),
- 100 veh/hr turning left in the peak hour, and
- 4000 ADT on the minor road.

The amount of expected delay reduction that would be attributed to the left-turn lane during the peak hour is 0.16 sec/veh. This value was determined using the regression coefficients; however, it could also be determined from Figure 23. The amount of delay savings per traffic period is shown in Table 72. For this site, the annual delay saving is $816 assuming a per-hour vehicle cost of $20.01.

Table 72. Calculation of annual delay savings for urban and suburban example.

Traffic Period	Hourly Factor	Hours/ Year	V_{TR} (veh/ hr)	V_{LT} (veh/ hr)	DR (sec/ veh)	DR (sec/Year)	DR (hr/Year)
AM Peak	10	522	325	100	0.16	63,663	17.68
PM Peak	10	522	325	100	0.16	63,663	17.68
Off Peak/ Weekend Peak	6.1	2243	198	61	0.01	10,262	2.85
Evening	2.8	2555	91	28	0.01	5,366	1.49
Night	1.8	2920	59	18	0.01	3,942	1.10
Total Hours/Year of Delay Reduction							40.80
Dollars per Vehicle Hour Cost							$20.01
Annual Delay Savings							$816.44

V_{TR} = Through and right-turn volume on major approach
V_{LT} = Left-turn volume
DR = Delay reduction

Annual Crash Savings

The determination of the savings attributed to crashes begins with calculating the predicted number of crashes for the intersection. For urban or suburban intersections, the predicted number of crashes consists of the predicted number of multiple-vehicle crashes, predicted number of single-vehicle crashes, predicted number of pedestrian-vehicle crashes, and predicted number of bicycle-vehicle crashes both before and after a left-turn lane is installed. This study is interested in the number of crashes that did not occur due to the addition of the left-turn lane. The equations are shown in Table 50. Following is an illustration of the steps for this example:

$$\begin{aligned} N_{spf\ U/S\text{-}MV,\ 3st} &= \exp[-13.36 + 1.11 \times \ln(AADT_{maj}) + 0.41 \times \ln(AADT_{min})] \\ &= \exp[-13.36 + 1.11 \times \ln(14{,}000) + 0.41 \times \ln(4000)] \\ &= 1.89 \text{ multiple-vehicle crashes/year} \end{aligned} \quad (58)$$

$$\begin{aligned} N_{spf\ U/S\text{-}SV,\ 3st} &= \exp[-6.81 + 0.16 \times \ln(AADT_{maj}) + 0.51 \times \ln(AADT_{min})] \\ &= \exp[-6.81 + 0.16 \times \ln(14{,}000) + 0.51 \times \ln(4000)] \\ &= 0.35 \text{ single-vehicle crashes/year} \end{aligned} \quad (59)$$

$$\begin{aligned} N_{spf\ U/S,\ 3st,\ M\&S,\ bef} &= (N_{spf\ U/S\text{-}MV,\ 3st} + N_{spf\ U/S\text{-}SV,\ 3st}) \\ &= (1.89 + 0.35) = 2.24 \text{ crashes/year} \end{aligned} \quad (60)$$

$$\begin{aligned} N_{spf\ U/S\text{-}Ped,\ 3st,\ bef} &= 0.021 \times (N_{spf\ U/S,\ 3st,\ M\&S,\ bef}) \\ &= 0.021 \times (2.24) = 0.05 \end{aligned} \quad (61)$$

$$\begin{aligned} N_{spf\ U/S\text{-}Bike,\ 3st,\ bef} &= 0.016 \times (N_{spf\ U/S,\ 3st,\ M\&S,\ bef}) \\ &= 0.016 \times (2.24) = 0.04 \end{aligned} \quad (62)$$

$$\begin{aligned} N_{spf\ U/S,\ 3st,\ bef} &= N_{spf\ U/S,\ 3st,\ M\&S,\ bef} + N_{spf\ U/S\text{-}Ped,\ 3st,\ bef} + N_{spf\ U/S\text{-}Bike,\ 3st,\ bef} \\ &= 2.24 + 0.05 + 0.04 = 2.32 \end{aligned} \quad (63)$$

$$\begin{aligned} N_{spf\ U/S,\ 3st,\ M\&S,\ aft} &= (N_{spf\ U/S\text{-}MV,\ 3st} + N_{spf\ U/S\text{-}SV,\ 3st}) \times AMF_{LTL} \\ &= (1.89 + 0.35) \times 0.67 = 1.50 \text{ crashes/year} \end{aligned} \quad (64)$$

$$\begin{aligned} N_{spf\ U/S\text{-}Ped,\ 3st,\ aft} &= 0.021 \times (N_{spf\ U/S,\ 3st,\ M\&S,\ aft}) \\ &= 0.021 \times (1.50) = 0.03 \end{aligned} \quad (65)$$

$$\begin{aligned} N_{spf\ U/S\text{-}Bike,\ 3st,\ aft} &= 0.016 \times (N_{spf\ U/S,\ 3st,\ M\&S,\ aft}) \\ &= 0.016 \times (1.50) = 0.02 \end{aligned} \quad (66)$$

$$\begin{aligned} N_{spf\ U/S,\ 3st,\ aft} &= N_{spf\ U/S,\ 3st,\ M\&S,\ aft} + N_{spf\ U/S\text{-}Ped,\ 3st,\ aft} + N_{spf\ U/S\text{-}Bike,\ 3st,\ aft} \\ &= 1.50 + 0.03 + 0.02 = 1.56 \end{aligned} \quad (67)$$

$$\begin{aligned} N_{U/S,\ 3st,\ pred\text{-}saved} &= N_{spf\ U/S,\ 3st,\ bef} - N_{spf\ U/S,\ 3st,\ aft} \\ &= 2.32 - 1.56 = 0.767 \text{ crashes not occurring/year} \end{aligned} \quad (68)$$

The costs per crash at three-leg urban and suburban intersections are:

- $167,000 (mid-range),
- $92,000 (low range), and
- $242,000 (high range).

Multiplying the cost per crash by the number of crashes that would not occur due to the presence of the left-turn lane results in the following annual crash savings:

- $128,062 (average),
- $70,549 (low range), and
- $185,574 (high range).

Present Worth of Annual Savings and Benefit-Cost Ratio

The delay and crash savings above represent the expected value per year. Assuming a 20-year design life and a 4 percent return, those annual savings need to be converted to a present worth so that the expected construction cost of the left-turn lane can be considered. The construction cost was estimated to be $250,000.

The B/C ratio when using the mid-range societal cost is:

$$\frac{B}{C} = \frac{13.59 \times (\$816 + \$128{,}062)}{\$250{,}000} = 7.0 \qquad (69)$$

The B/C ratio of 7.0 indicates that the installation of a left-turn lane should be warranted for these conditions.

GREEN BOOK **WARRANTS FOR LEFT-TURN LANE**

Figure 27 shows a plot of the *Green Book* rural two-lane highway left-turn warrants in a graphical form that uses left-turn lane volume and average peak-hour major-road approach volume. The current *Green Book* warrant is based on percent left turns and uneven approach lane volumes, hence the multiple left-turn volume for a given major-road volume in Figure 27. For a simpler showing of the *Green Book* data in other graphs, lines representing a subset of the *Green Book* data were generated for each speed (see Figure 28).

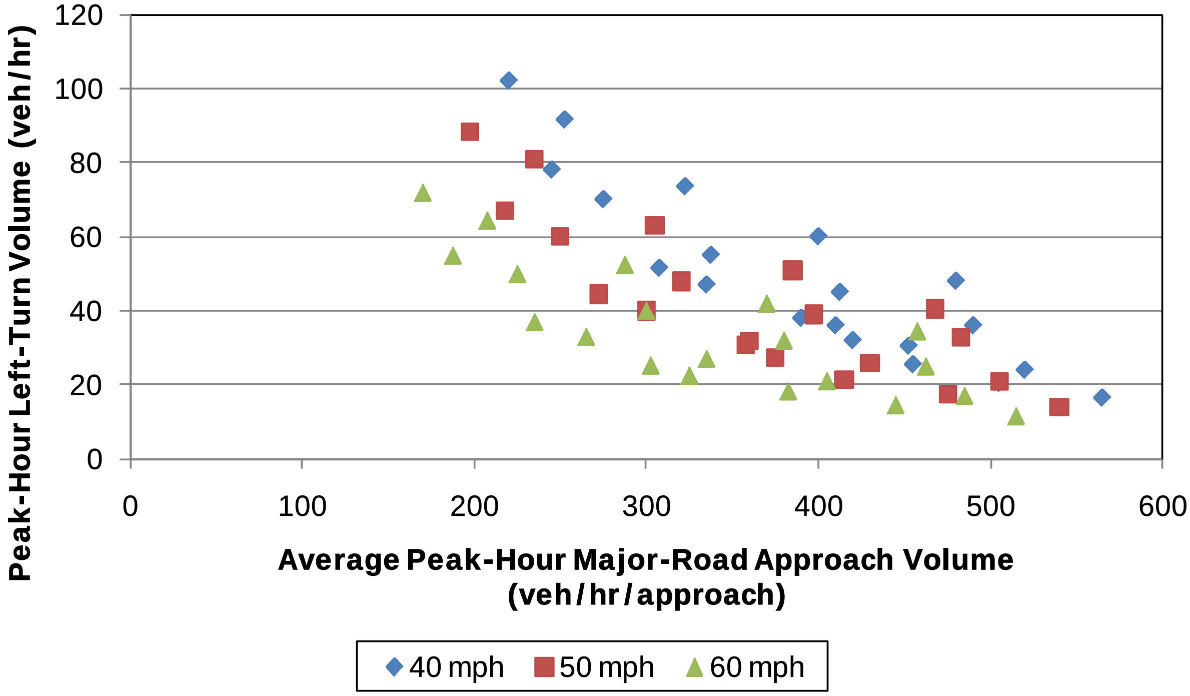


Figure 27. Plot of *Green Book* rural two-lane highway left-turn warrant values.

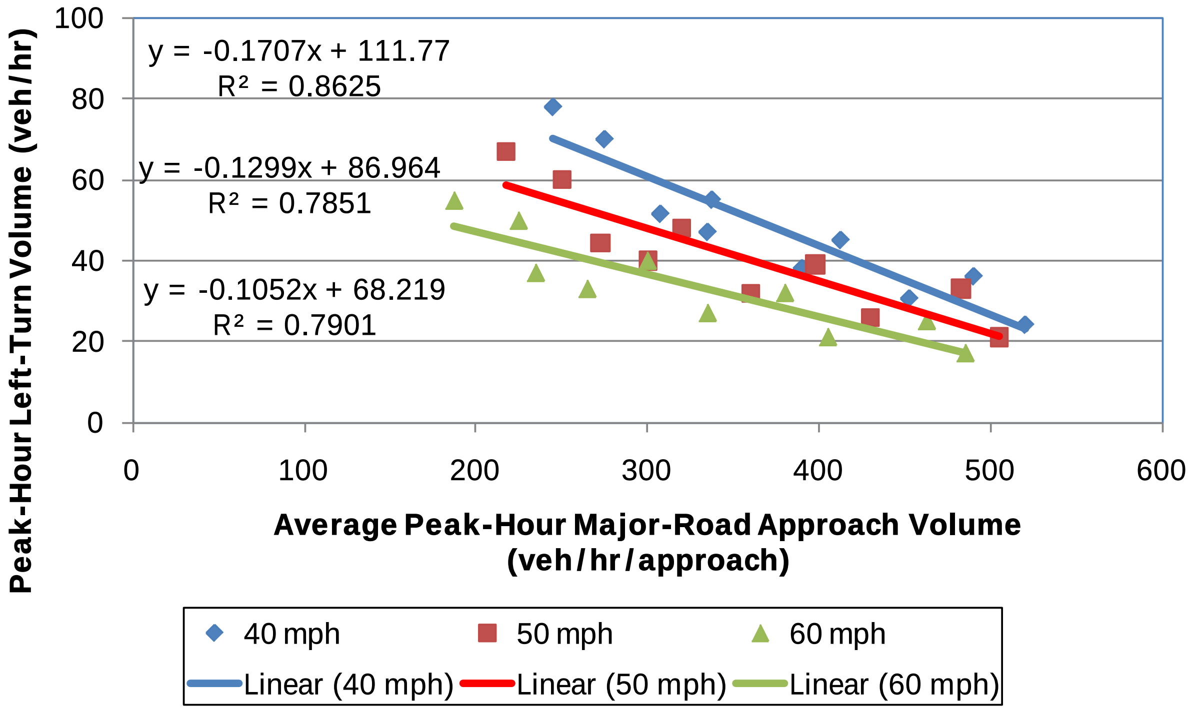


Figure 28. Lines generated to represent a subset of the *Green Book* rural two-lane highway left-turn warrant values.

DEVELOP PRELIMINARY WARRANTS FOR LEFT-TURN LANE

The benefit-cost methodology presented above was applied to a range of major-road ADTs (1000 to 15,000) and minor-road ADTs (200 to 3000) along with a range of posted speed limits (30 to 60 mph). A service life of 20 years and a minimum rate of return of 4 percent were assumed. The minimum major-road ADT for a given left-turn lane volume that gave a B/C of 1.0 and 2.0 was identified. Initially, the minimum major-road ADT was determined for each posted speed limit; however, minimal differences were identified, primarily because crash costs dominated the calculations and crash prediction is not a function of posted speed limit. Crash predictions are different for rural and urban conditions; therefore, the results will be presented uniquely for rural and urban conditions. Crash predictions also vary by the number of legs at the intersection; therefore, results will also be presented for three-leg and four-leg intersections.

The FHWA memo on the economic value of a statistical life recommends that a range be considered in an analysis. Therefore, researchers used the following values (and abbreviations in later figures) in the analysis:

- Mid-range = $5.8 million (mid crash),
- Low range = $3.2 million (low crash), and
- High range = $8.4 million (high crash).

The cost to install a left-turn lane can vary from a minimal amount such as the cost to restripe a roadway section to a very large value that would occur if right-of-way needs to be purchased in addition to the construction costs. The range of construction costs selected for the analysis and the abbreviations used to describe the costs in later figures are as follows:

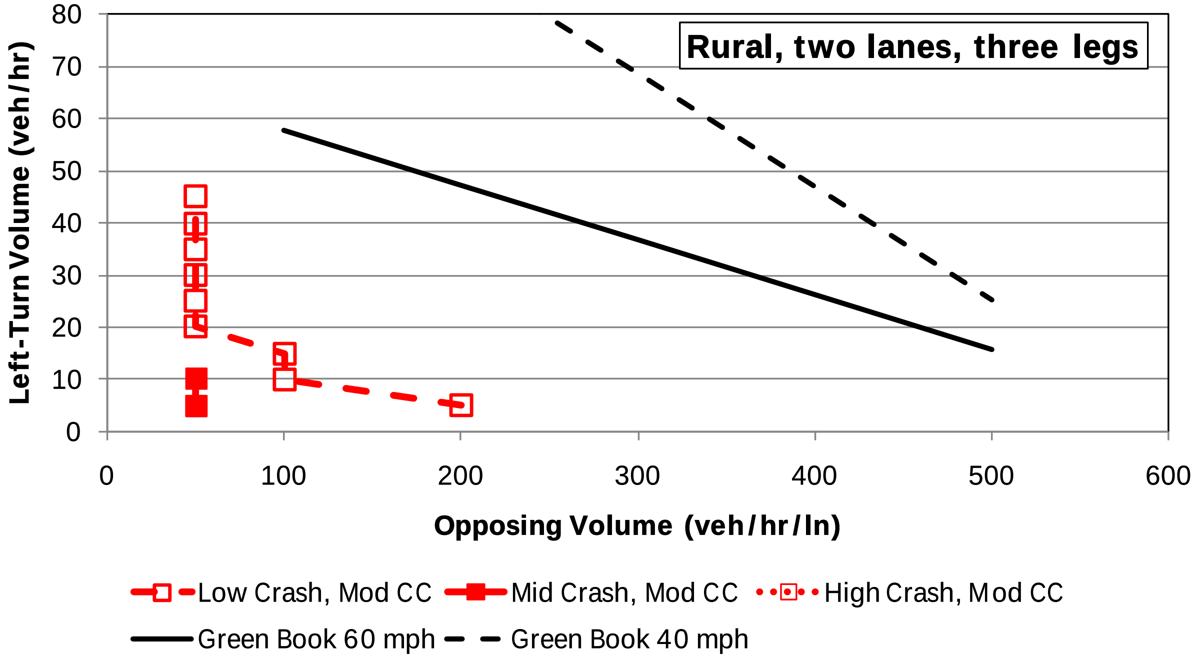
- $100,000 (min CC for minimum construction costs),
- $250,000 (mod CC for moderate construction costs), and
- $375,000 (max CC for maximum construction costs).

Plots of Preliminary Warrants

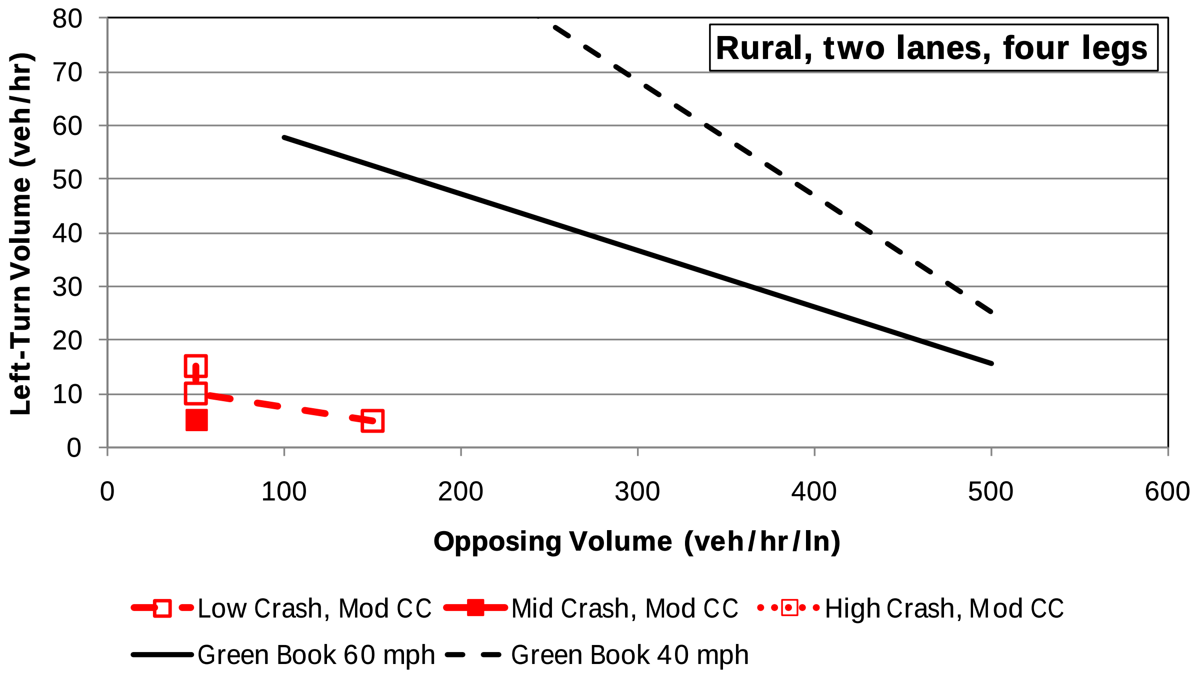
Plots were developed to illustrate the resulting warrants for a range of assumed statistical life values and construction costs. The plots selected for illustration in this report are shown in:

- Figure 29 for rural two-lane highways showing a range of crash costs (based on $3.2, $5.8, and $8.4 million value of a statistical life),
- Figure 30 for rural four-lane highways showing a range of crash costs (based on $3.2, $5.8, and $8.4 million value of a statistical life),
- Figure 31 for urban and suburban arterials showing a range of crash costs (based on $3.2, $5.8, and $8.4 million value of a statistical life),
- Figure 32 for rural two-lane highways showing a range of construction costs ($100,000, $250,000, and $375,000),
- Figure 33 for rural four-lane highways showing a range of construction costs ($100,000, $250,000, and $375,000), and
- Figure 34 for urban and suburban arterials showing a range of construction costs ($100,000, $250,000, and $375,000).

The figures for rural two-lane highways or urban and suburban arterials also include a representation of the current *Green Book* warrants.

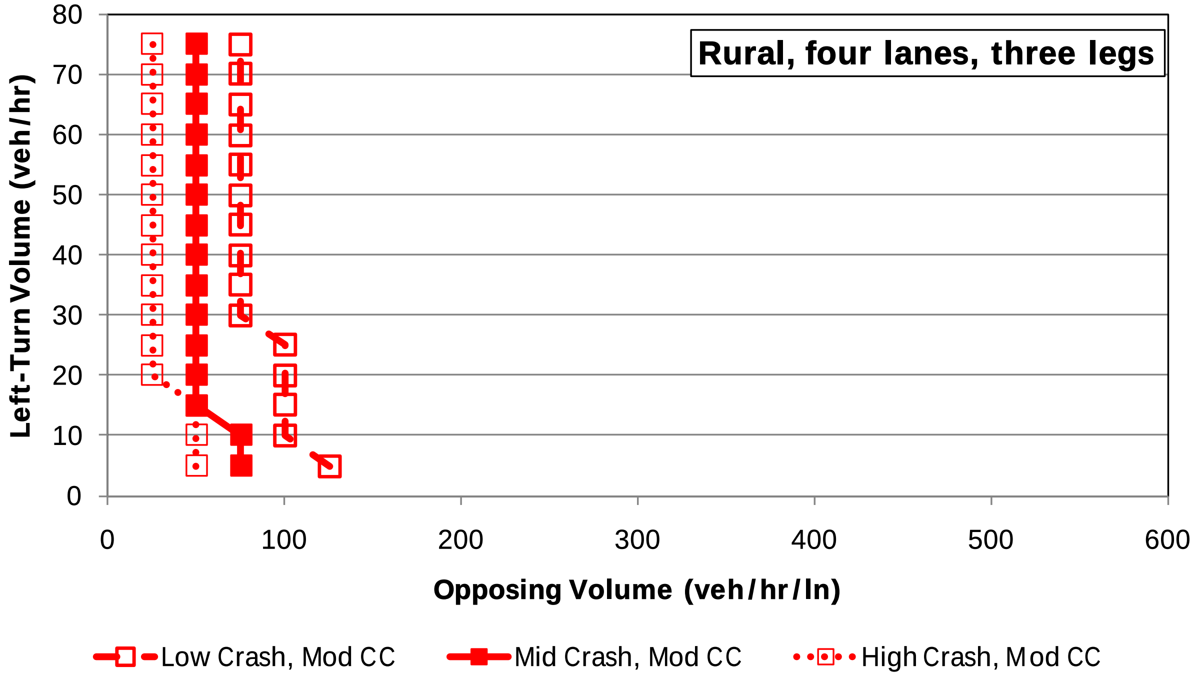


Two lanes, three legs

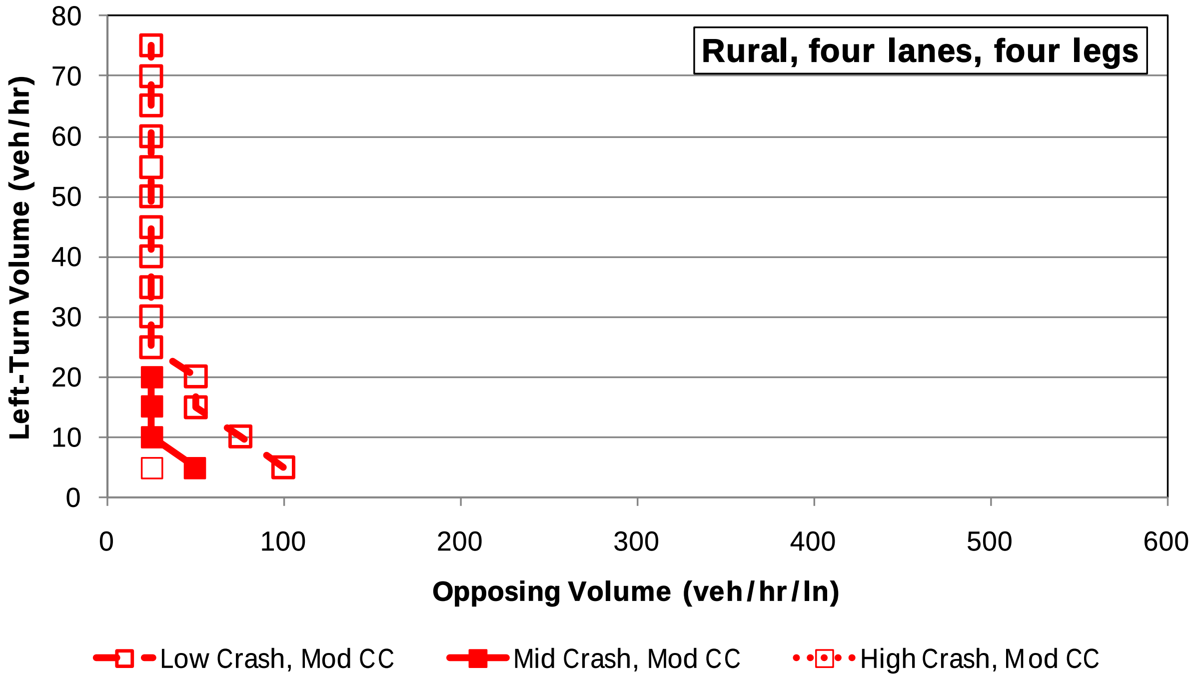


Two lanes, four legs

Figure 29. Range of left-turn lane warrants based on crash costs (low, mid-, and high range) for rural two-lane highway.

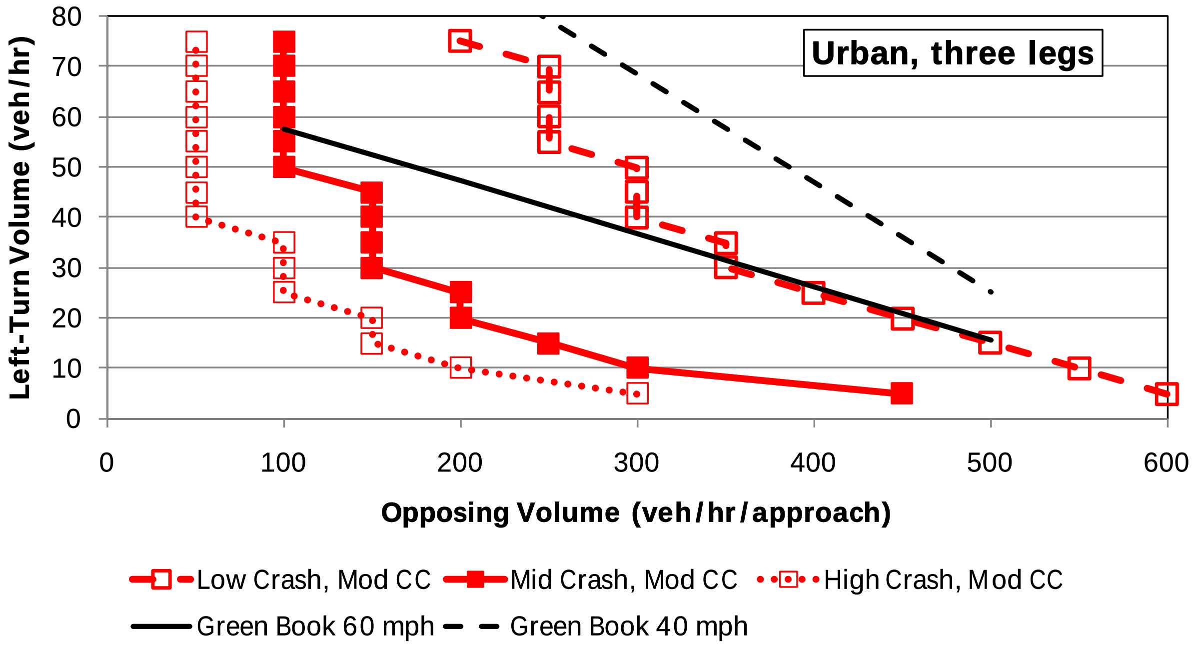


Four lanes, three legs

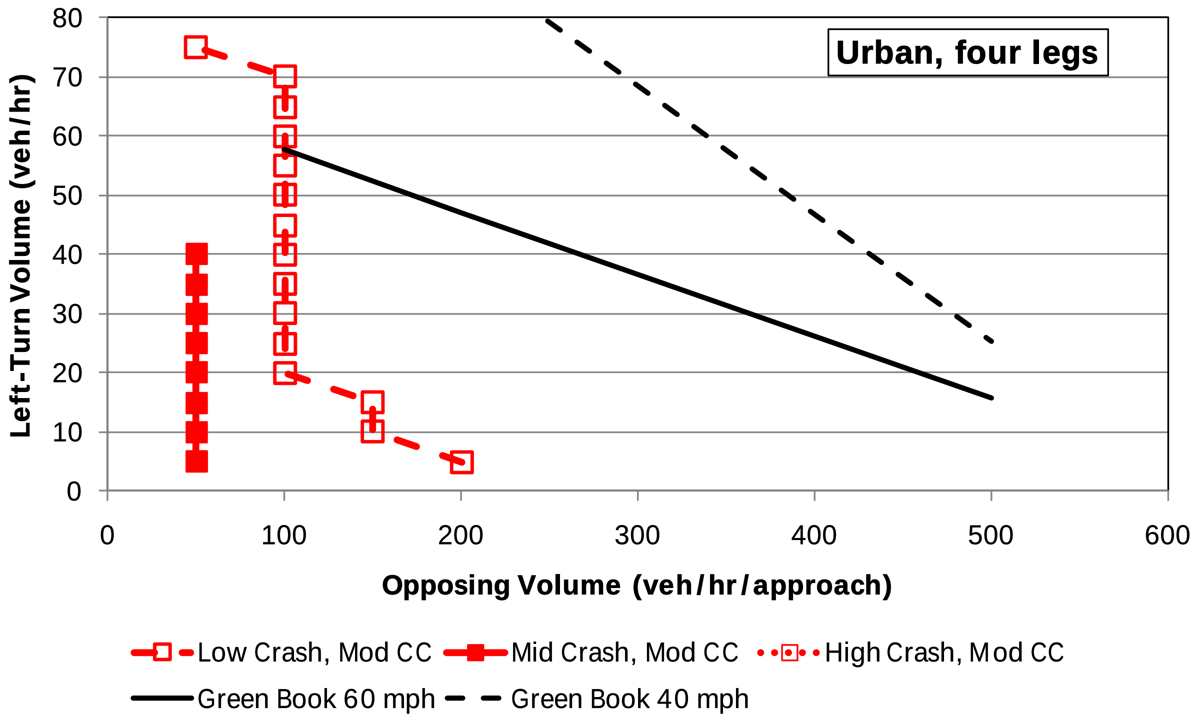


Four lanes, four legs

Figure 30. Range of left-turn lane warrants based on crash costs (low, mid-, and high range) for rural four-lane highway.

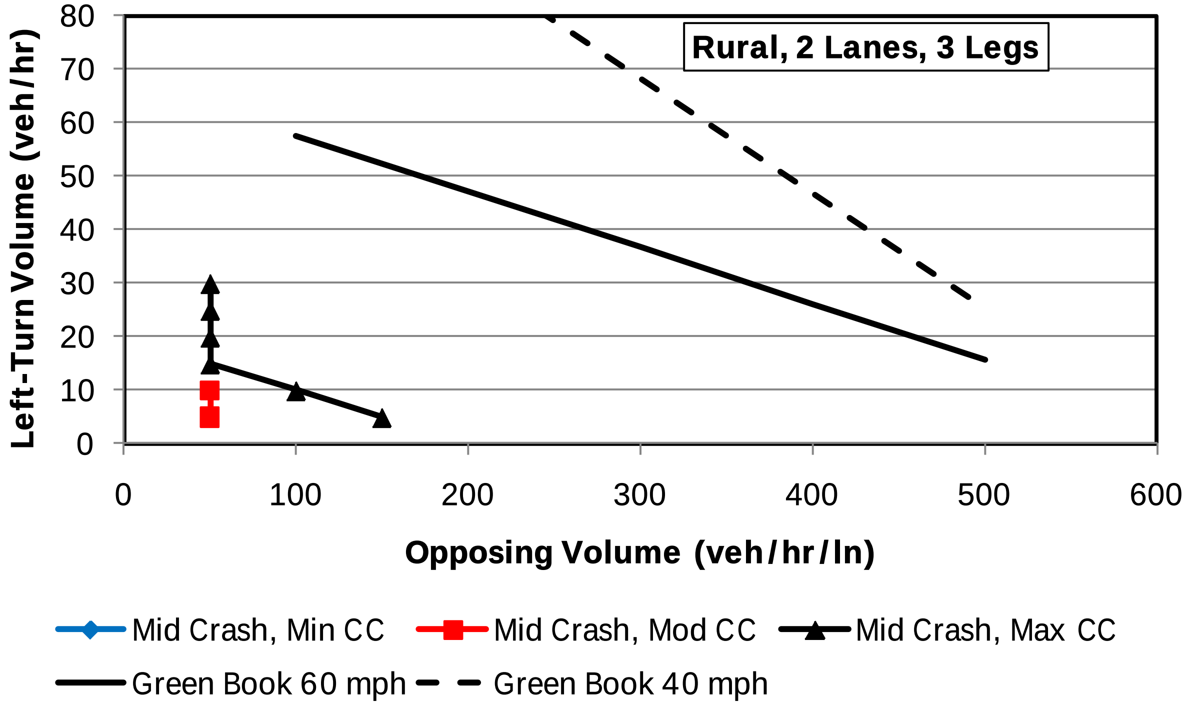


Three legs

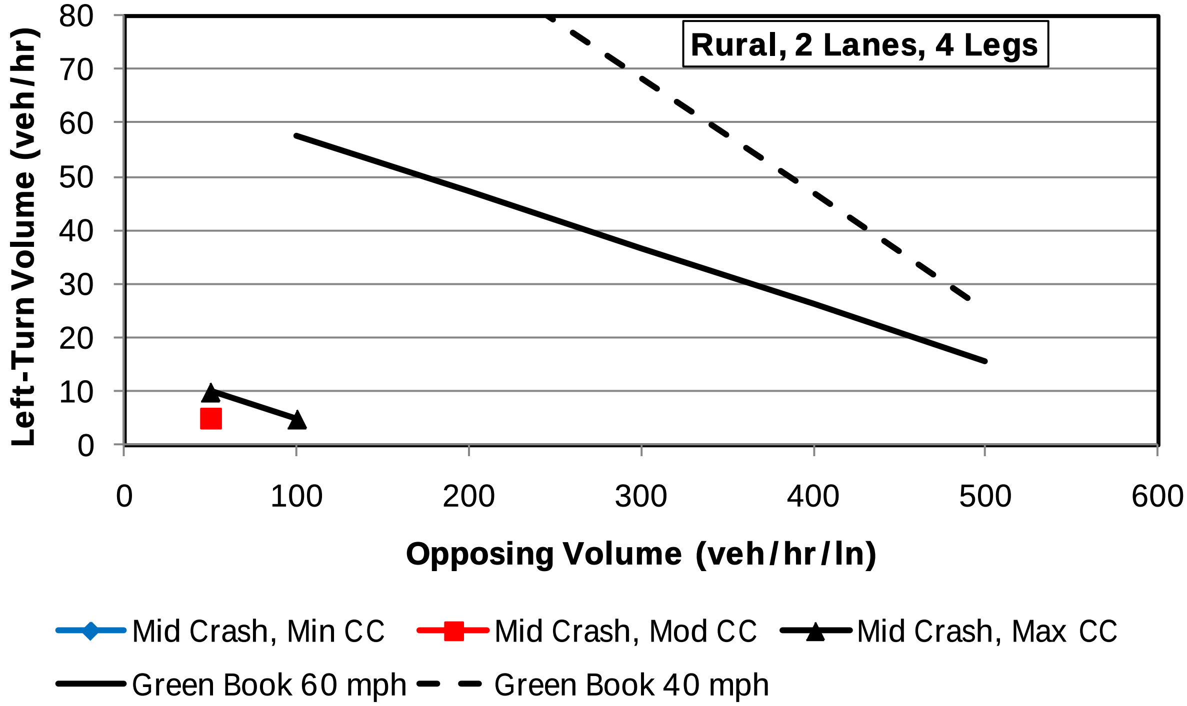


Four legs

Figure 31. Range of left-turn lane warrants based on crash costs (low, mid-, and high range) for urban and suburban highways.

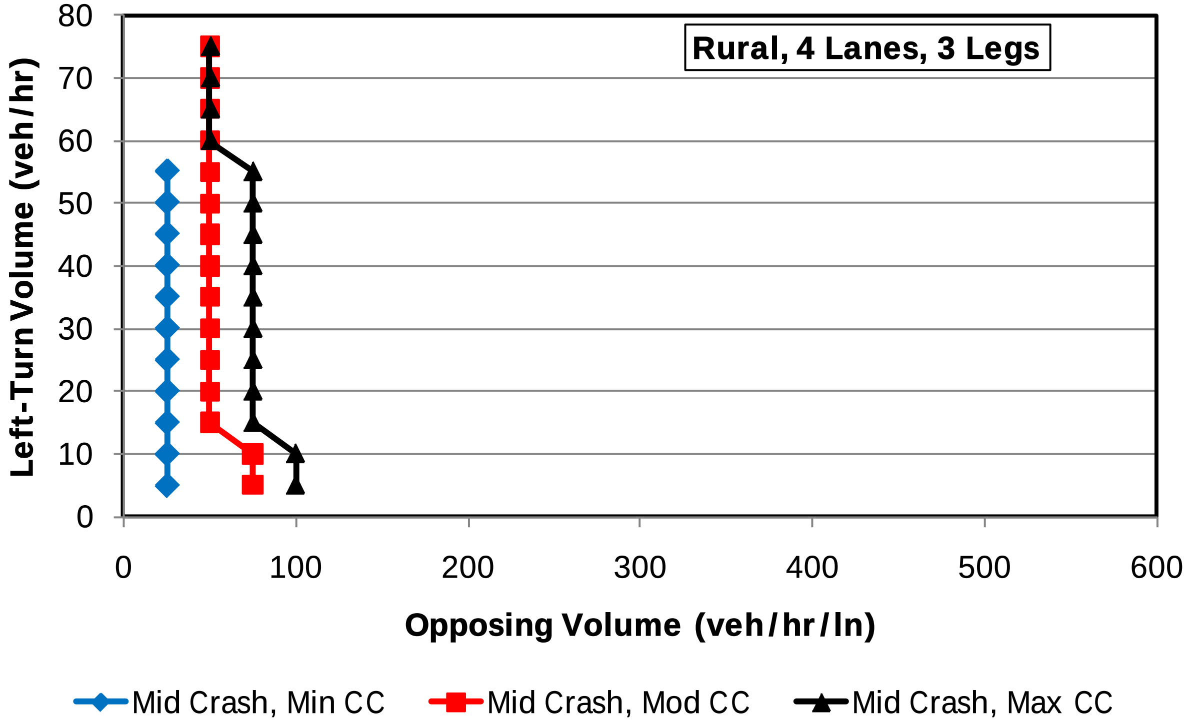


Two lanes, three legs

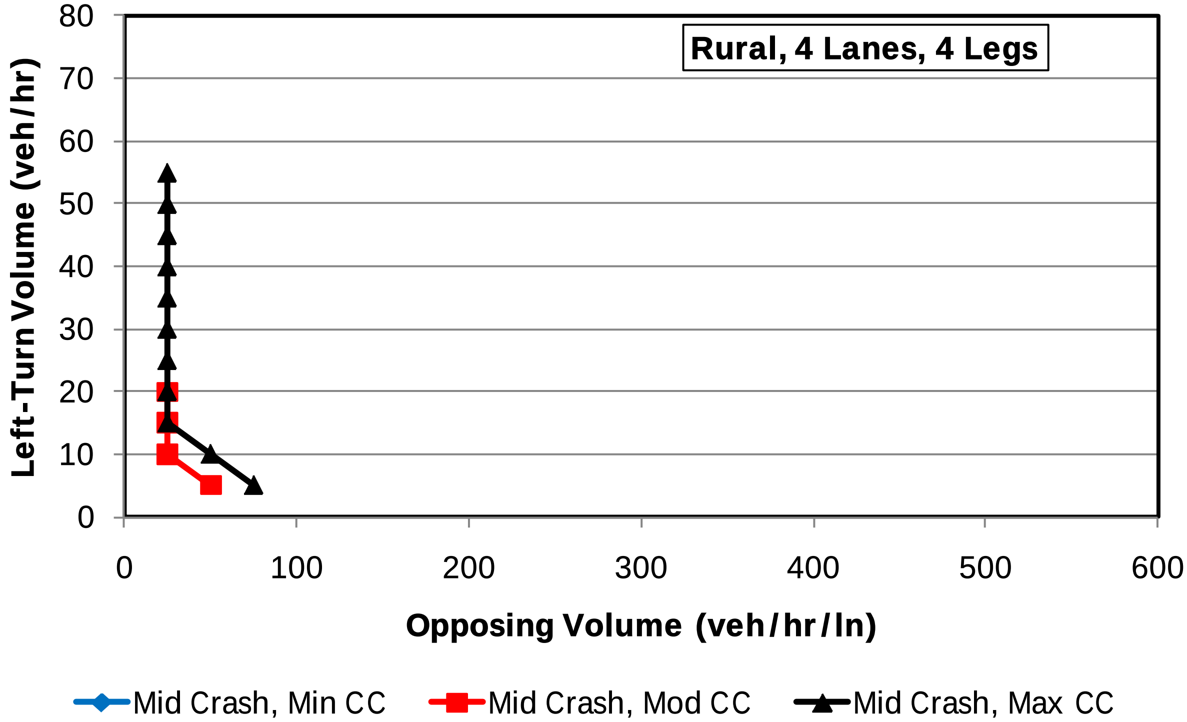


Two lanes, four legs

Figure 32. Range of left-turn lane warrants based on construction costs (minimum, moderate, and maximum) for rural two-lane highways.

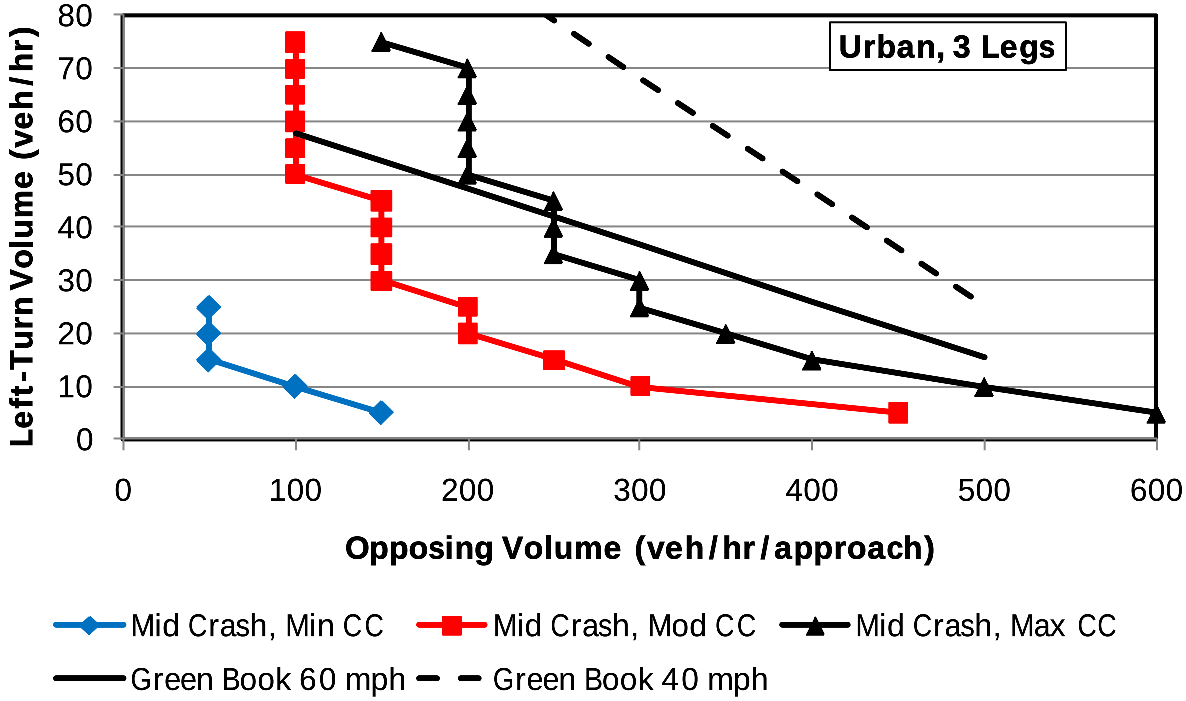


Four lanes, three legs

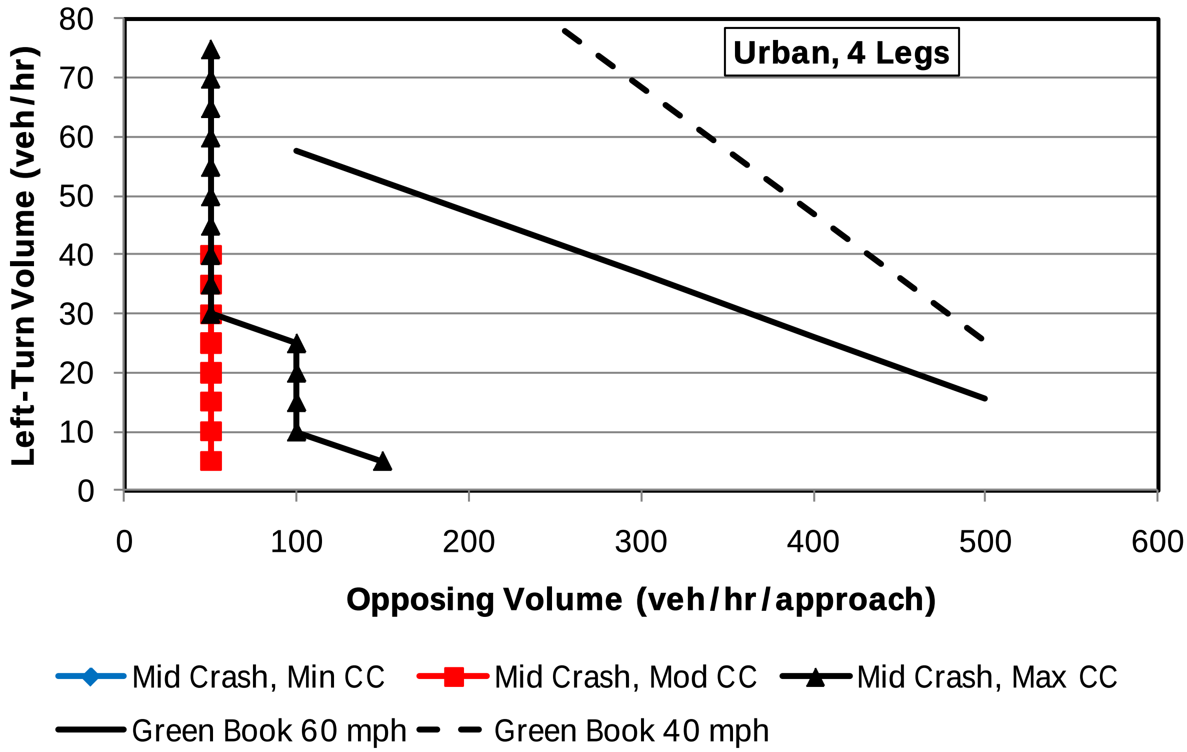


Four lanes, four legs

Figure 33. Range of left-turn lane warrants based on construction costs (minimum, moderate, and maximum) for rural four-lane highways.



Three legs



Four legs

Figure 34. Range of left-turn lane warrants based on construction costs (minimum, moderate, and maximum) for urban and suburban arterials.

The lines shown in the plots represent the minimum volumes for which a left-turn lane would be recommended based on anticipated delay reduction and crash reduction from adding a left-turn lane to an existing site. In each plot, the point corresponding to the peak-hour major-road volume and the peak-hour left-turn volume should be located. If the point is to the right of the curve shown for the number of lanes and number of legs, then provision of a left-turn lane would be economically justified (i.e., its B/C ratio would be greater than 1.0). If the point is to the left of the curve shown for the appropriate number of lanes and legs, then provision of a left-turn lane is not economically justified (i.e., its B/C ratio would be less than 1.0).

Observations Regarding Preliminary Warrants

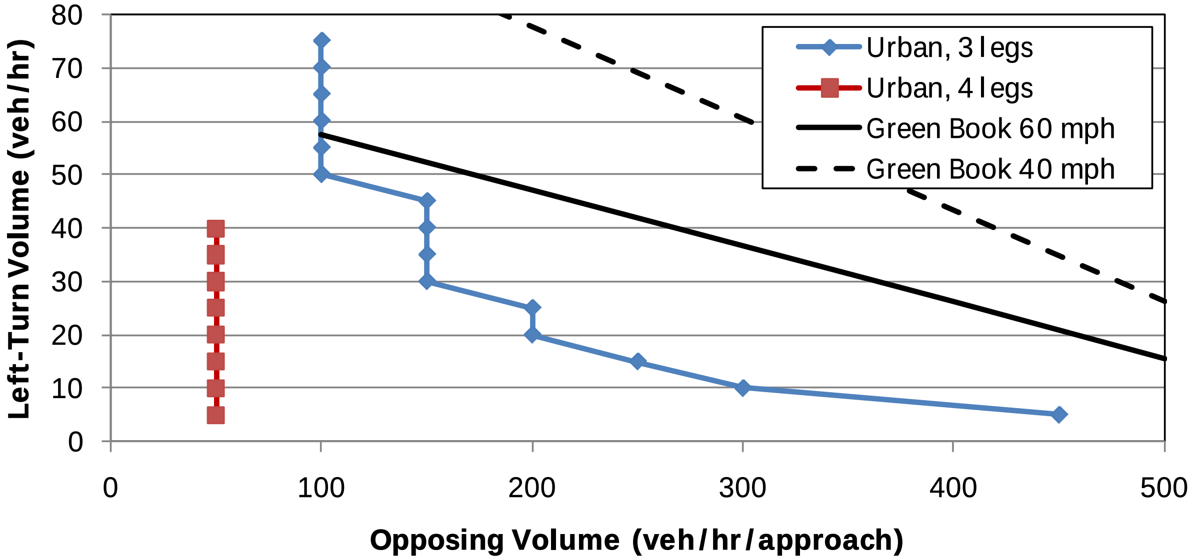
Observations from the graphs include:

- Comparing the range of warrants resulting from the range of construction costs and the range of crash costs, much less variability in warrants results when the construction costs are variable by the amounts used in this analysis ($100,000 to $375,000) as can be seen in Figure 32, Figure 33, and Figure 34. A greater range is seen when the crash costs are varied (as shown in Figure 29, Figure 30, and Figure 31).
- The warrants for two-lane highways are noticeably smaller than the existing *Green Book* warrants (as shown in Figure 29).
- Left-turn lanes are always warranted on four-leg intersections at lower volumes as compared to three-leg intersections.
- Left-turn lanes are warranted on rural two-lane highways at very low volumes (shown in Figure 29). As few as five left-turning vehicles crossing 50 veh/hr/ln results in benefits that outweigh costs when using low- or mid-range crash costs. Even with using high-range crash costs, the benefits are greater than the costs when the five left-turning vehicles are crossing as few as 150 veh/hr/ln for four-leg intersections or 200 veh/hr/ln for three-leg intersections.
- Three-leg intersections on urban and suburban arterials have the greatest range in warrant values. Using mid-range crash costs, the warrants range from 50 left-turning vehicles and 100 veh/hr/ln on the major road to five left-turning vehicles and 450 veh/hr/ln on the major road. These warrants are less than the current *Green Book* warrants. If a low crash cost is assumed, some of the resulting warrants are higher than some of the *Green Book* warrants (see Figure 31).

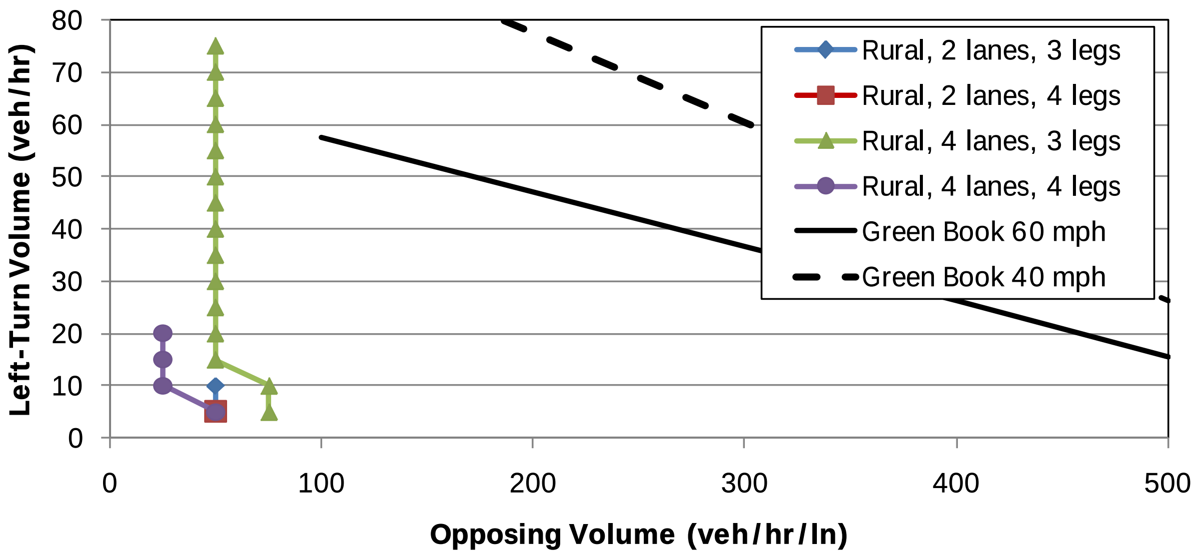
The primary observation from the graphs that compare the ranges in value for statistical life and construction cost is that the variations in assumed crash cost have a greater impact on the resulting warrants than the variations in assumed construction cost.

Other Preliminary Warrants

Another approach suggested to account for differences in costs is to utilize a benefit-cost ratio that is higher than 1.0. Plots were developed to illustrate the difference between assuming a benefit-cost ratio of 1.0 (see Figure 35) as compared to 2.0 (see Figure 36) using mid-range crash cost and moderate construction cost. Table 73 provides the warrants in a tabular form. It shows the major-road peak-hour volume that would warrant a left-turn lane for a given left-turn volume using a benefit-cost ratio of 1.0 and 2.0.

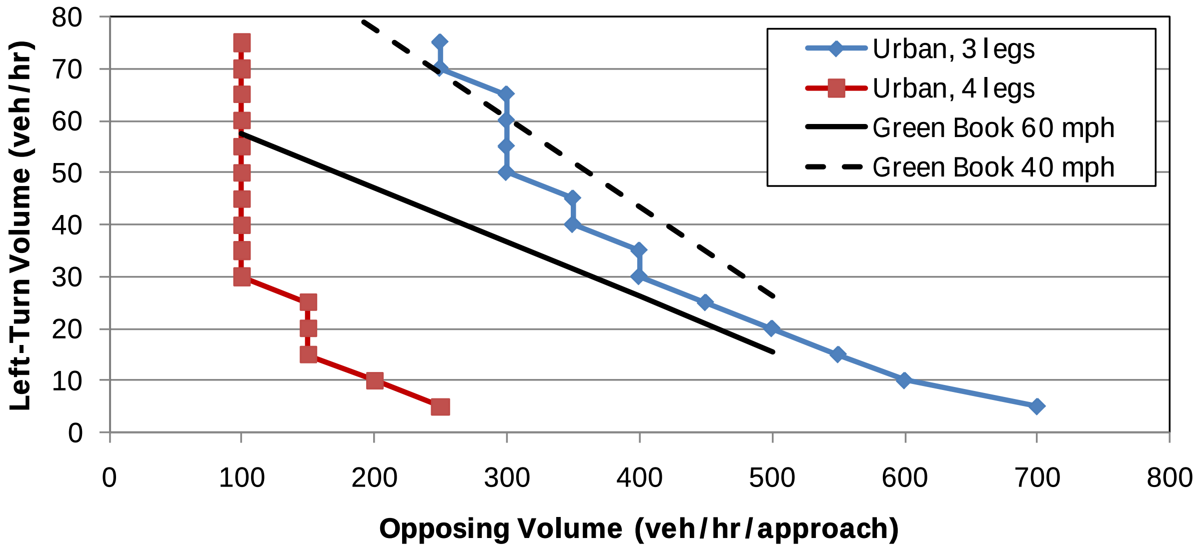


Suggested left-turn warrants for urban and suburban arterials

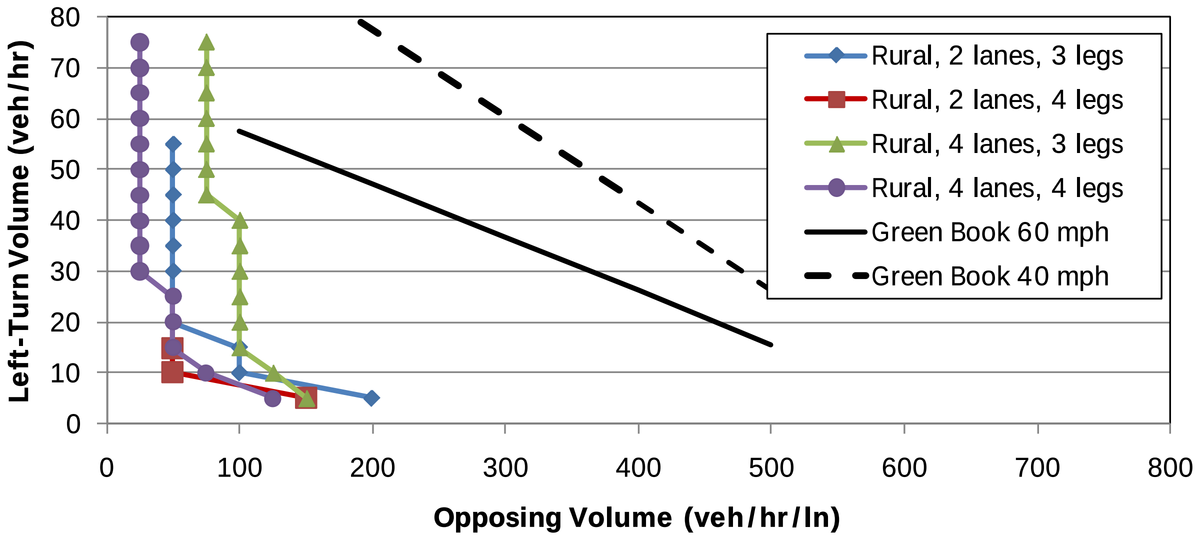


Suggested left-turn warrants for rural highways

Figure 35. Suggested left-turn warrants based on results from benefit-cost evaluation when using B/C of 1.0 and mid-range crash cost and moderate construction cost.



Suggested left-turn warrants for urban and suburban arterials



Suggested left-turn warrants for rural highways

Figure 36. Suggested left-turn warrants based on results from benefit-cost evaluation when using B/C of 2.0 and mid-range crash cost and moderate construction cost.

Table 73. Range of left-turn lane warrants based on results from benefit-cost evaluations using crash costs developed based on FHWA economic value of a statistical life.

B/C Ratio	Peak-Hour Left-Turn Lane Volume (veh/hr)	Peak-Hour Major-Road Volume (veh/hr/ln) Based on Crash Costs Developed Using FHWA Economic Value of a Statistical Life							
		Two Lanes				Four Lanes			
		Three legs		Four legs		Three legs		Four legs	
		Rural	Urban	Rural	Urban	Rural	Urban	Rural	Urban
1.0	5	50	450	50	50	75	225	50	25
	10	50	300	<50	50	75	150	25	25
	15	<50	250	<50	50	50	125	25	25
	20	<50	200	<50	50	50	100	25	25
	25	<50	200	<50	50	50	100	<25	25
	30	<50	150	<50	50	50	75	<25	25
	35	<50	150	<50	50	50	75	<25	25
	40	<50	150	<50	50	50	75	<25	25
	45	<50	150	<50	<50	50	75	<25	<25
	50 or more	<50	100	<50	<50	50	50	<25	<25
2.0	5	200	700	150	250	150	375	125	125
	10	100	600	50	200	125	350	75	100
	15	100	550	50	150	100	275	50	75
	20	50	500	<50	150	100	250	50	75
	25	50	450	<50	150	100	225	50	75
	30	50	400	<50	100	100	200	25	50
	35	50	400	<50	100	100	200	25	50
	40	50	350	<50	100	100	175	25	50
	45	50	350	<50	100	75	175	25	50
	50	50	300	<50	100	75	150	25	50
	55	50	300	<50	100	75	150	25	50
	60	<50	300	<50	100	75	150	25	50
	65	<50	300	<50	100	75	150	25	50
	70 or more	<50	250	<50	100	75	125	25	50

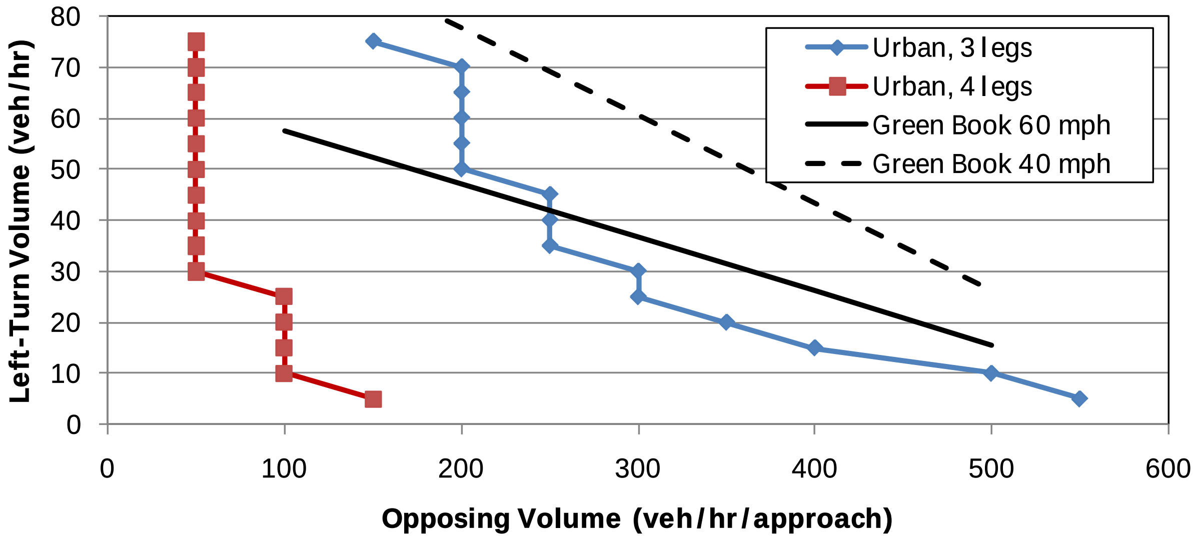
A greater difference in warrants can be seen for the urban and suburban arterial condition as compared to the rural condition. Even with a benefit-cost ratio of 2.0, the resulting warrants are less than the current *Green Book* warrants for rural highways (see Figure 36). For four-leg intersections on urban and suburban arterials, a turning volume of 30 veh/hr or more would warrant a left-turn lane when the major-road traffic is 100 veh/hr/approach. The higher benefit-cost ratio did not change that warrant. For lower left-turning volumes, the higher benefit-cost assumption did result in needing a higher peak-hour major-road volume before warranting a left-turn lane. Much higher volumes would need to be met to warrant a left-turn lane on an urban and suburban three-leg intersection (as can be seen when comparing Figure 35 to Figure 36).

Since the *Highway Safety Manual* equations are used, some may argue that the *Highway Safety Manual* costs should also be used. The warrants resulting from assuming the *Highway Safety Manual* crash costs (adjusted to 2009 dollars) are listed in Table 74 and illustrated in Figure 37. The resulting warrants are lower when using the *Highway Safety Manual* costs as compared to using a B/C ratio of 2.0 with the mid-range crash costs and moderate construction costs (see Figure 36).

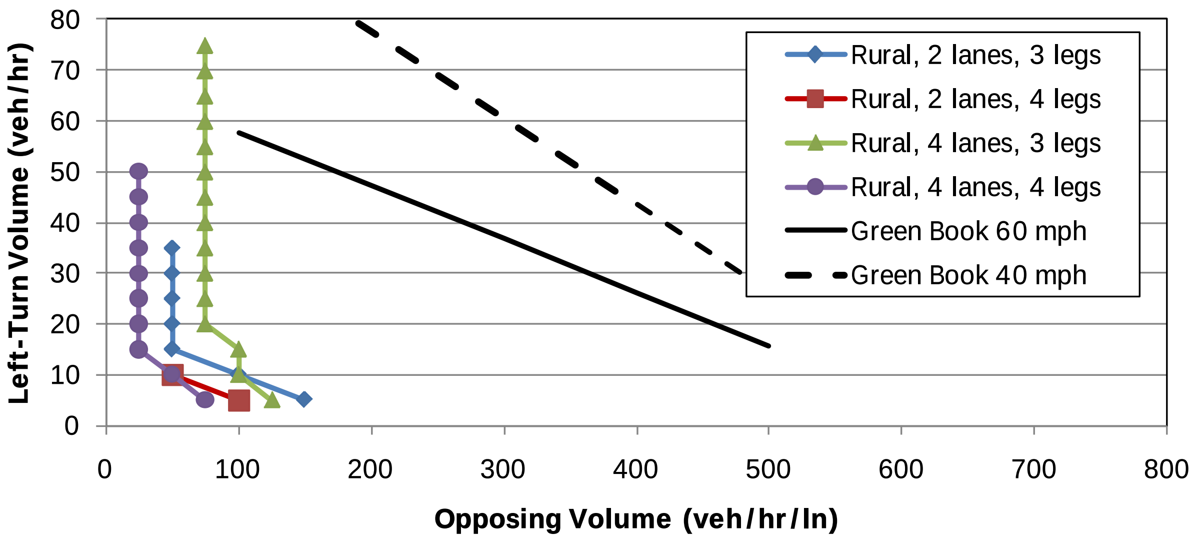
The *Highway Safety Manual* prediction equations and crash costs provide sensitivity to the number of crashes in the rural versus urban conditions. The crash costs, however, do not provide sensitivity to the number of injuries and deaths for a crash in an urban area as compared to a rural area. Future research should determine how the number of vehicles involved in a crash varies between the rural and urban areas when using a larger dataset than what is available in this study.

Table 74. Range of left-turn lane warrants based on results from benefit-cost evaluations using crash costs developed from *Highway Safety Manual* crash costs.

B/C Ratio	Peak-Hour Left-Turn Lane Volume (veh/hr)	Peak-Hour Major-Road Volume (veh/hr/ln) Based on *Highway Safety Manual* Crash Costs							
		Two Lanes				Four Lanes			
		Three Legs		Four Legs		Three Legs		Four Legs	
		Rural	Urban	Rural	Urban	Rural	Urban	Rural	Urban
1.0	5	150	550	100	150	125	350	75	75
	10	100	500	50	100	100	250	50	50
	15	50	400	<50	100	100	200	25	50
	20	50	350	<50	100	75	175	25	50
	25	50	300	<50	100	75	150	25	50
	30	50	300	<50	50	75	150	25	25
	35	50	250	<50	50	75	125	25	25
	40	<50	250	<50	50	75	125	25	25
	45	<50	250	<50	50	75	125	25	25
	50	<50	200	<50	50	75	100	25	25
	55	<50	200	<50	50	75	100	<25	25
	60	<50	200	<50	50	75	100	<25	25
	65	<50	200	<50	50	75	100	<25	25
	70	<50	200	<50	50	75	100	<25	25
	75	<50	150	<50	50	75	75	<25	25



Suggested left-turn warrants for urban and suburban arterials



Suggested left-turn warrants for rural highways

Figure 37. Suggested left-turn warrants based on results from benefit-cost evaluation when using B/C of 1.0 and HSM crash costs (2009 dollars).

IMPACTS DUE TO A NEW DEVELOPMENT

There is an inherent difference between adding a left-turn lane at an existing site and constructing a new left-turn lane at a proposed development. This difference was reflected in the explanation of the computation of delay for an existing site and for a new development.

At an existing site, the fundamental question is whether a left-turn lane should be provided to alleviate operational and/or safety issues that may be related to the absence of a left-turn lane. In other words, what is the tradeoff between the benefits that may be achieved and the costs of providing a left-turn lane? For the installation of a left-turn lane at a new development, there are additional questions to answer.

As part of its responsibility to manage a roadway network, a transportation agency must decide how/where to spend its limited resources to achieve the maximum benefit to the public. As a result, a process is needed to establish a ranking for which locations should receive priority for having left-turn accommodations provided. The benefit-cost approach enables an agency to make decisions based on estimates of the benefits—in terms of delays and crashes—and costs. Due to limited resources, an agency may often be forced to accept operations and safety conditions that it would have wanted to prevent from occurring. Access management—in particular, the driveway permitting process—is one approach for being proactive.

For access to a new development, state and local agencies typically use access permitting to apply access management standards to guide decisions regarding where and what access would be allowed as well as any restrictions to this access. The increasing demands for highway access make it increasingly clear that driveways, and the developments they serve, can have cumulative adverse impacts on the safety and efficiency of the roadway system.

While private property enjoys the right of access to the general system of public roadways, this is not an unlimited right. The right of access must be balanced with the needs of and potential harm to the general traveling public. In order to preserve mobility and provide safety for the traveling public, many transportation agencies have established regulations and programs to manage access to their roadway network. The regulations are more restrictive for major arterials, the roadways intended to accommodate higher volumes and speeds.

Access management programs restrict the number of driveways allowed as well as the movements allowed that are to be accommodated at the driveways. These practices affect when and where direct driveway access will be allowed onto the roadway network, whether alternative access should be provided, and the need for shared access. If direct access is allowed, the guidance includes the extent of that access (i.e., right in and right out versus full movement) and circumstances in which multiple driveways are allowed. In addition, agencies may require that steps be taken by a developer to mitigate projected traffic operations and/or safety impacts. An example of mitigation is providing a left-turn lane to remove vehicles turning left into the site from the through traffic lanes on an arterial.

Many transportation agencies have the authority to require a developer to pay for this mitigation as long as there is a rational nexus between the projected impacts of the development and the needed improvements. In this manner, taxpayers do not have to pay for an improvement that may benefit predominantly one property owner. Transportation agencies that do not have this authority may need to coordinate with the land use authorities to require that needed mitigation be provided at the developer’s expense.

Therefore, for a new development the first question for a transportation agency is whether access should be allowed at a particular location or would be preferable at an alternate location. The second question is, if access is allowed, what movements should be permitted. A follow-up question is, if the left-turn inbound movement is allowed, should a left-turn lane be provided, and who should pay for its construction.

Example of New Development

Additional Annual Delay

The findings from this study can also be used to estimate the additional costs to a site if a new development causes left-turn volumes to increase. This example is for the conditions assumed, such as no cross minor-road volumes for a four-leg intersection. Simulation for a specific site would be needed to determine the expected increase in delay for the given intersection characteristics.

Assume that the conditions at the site include the following:

- Two lanes,
- Rural location,
- Four legs,
- 60-mph posted speed limit,
- 488 veh/hr/ln in the peak hour (10,000 ADT on the major road),
- 25 veh/hr turning left in the peak hour,
- 1000 ADT on the minor road, and
- Left-turn lane added on one approach.

The amount of additional expected delay during the peak hour is 0.127 sec/veh. This value was determined using the regression coefficients; however, it could also be determined from Figure 24. The delay represents the per-vehicle delay. To determine the delay for all the vehicles at the intersection for that hour, the delay value needs to be multiplied by the number of vehicles at the intersection for that hour, in this example 1001 vehicles (488 through and right-turning vehicles per lane on each approach along with 25 left-turning vehicles). Therefore 66,063 sec of delay occurred during the AM peak hour. With 522 hours in that traffic period, the amount of delay for the year is 18.35 hours. The amount of delay per traffic period is shown in Table 75. For this site, the annual delay cost is $879 assuming a per-hour vehicle cost of $20.01.

Table 75. Calculation of annual delay for new development example.

Traffic Period	Hourly Percent	Hours/ Year	V_{TR} (veh/ hr)	V_{LT} (veh/ hr)	DR (sec/ veh)	DR (sec) in the Hour	DR (sec/Year)	DR (hr/Year)
AM Peak	10	522	488	25	0.127	127	66,063	18.35
PM Peak	10	522	488	25	0.127	127	66,063	18.35
Off Peak/ Weekend Peak	6.1	2243	297	15	0.010	6	13,682	3.80
Evening	2.8	2555	137	7	0.010	3	7,154	1.99
Night	1.8	2920	88	5	0.010	2	5,256	1.46
							158,218	43.95
Dollars per Vehicle Hour Cost								$20.01
Annual Delay								$879.43

Annual Crash Costs

The determination of the costs attributed to crashes begins with calculating the predicted number of crashes for the location. To determine the total number of crash for a roadway, the predicted number of crashes along the segment is added to the predicted number of crashes at the intersection(s). Since the interest within this exercise is to identify the additional crashes associated with this location when it becomes an intersection (now that it has left-turn volume demand due to the new development), using the HSM prediction equation for the intersection is appropriate. The assumption is that the predicted number of segment crashes at this location does not change in the two periods. What changes is the additional crashes estimated due to the location now having intersection-type traffic.

The equation for a rural two-lane three-leg intersection is:

$$N_{spf\ 2\ ln,\ 3st} = \exp[-8.56 + 0.60 \times \ln(AADT_{maj}) + 0.61 \times \ln(AADT_{min})] \quad (70)$$

$$N_{spf\ 2\ ln,\ 3st} = \exp[-8.56 + 0.60 \times \ln(10{,}000) + 0.61 \times \ln(1000)]$$

$$N_{spf\ 2\ ln,\ 3st} = 3.25 \text{ crashes/year}$$

The costs per crash at rural four-leg intersections are:

- $198,000 (average),
- $109,000 (low range), and
- $287,000 (high range).

Multiplying the cost per crash by the number of crashes that are predicted to occur since the segment now includes an intersection results in the following annual crash costs:

- $644,324 (average),
- $354,704 (low range), and
- $933,945 (high range).

Present Worth of Annual Costs

The delay and crash costs above represent the expected value per year. Assuming a 20-year design life and a 4 percent return, those annual costs can be converted to a present worth as follows:

$$Present\ Worth\ of\ Costs = 13.59\ \times (\$879 + \$644{,}324) = \$8{,}768{,}522 \tag{71}$$

CHAPTER 6

COMPARISON OF PROCEDURES

OVERVIEW OF IDENTIFIED PROCEDURES

Several states include warrants for the installation of left-turn lanes. These warrants are primarily based on:

- Left-turn volume (or percent of left turns),
- Opposing volume, and
- Approach volume.

Many warrants also suggest consideration of:

- Crashes,
- Sight distance,
- Operations (capacity analysis), or
- Functional classification of the major roadway.

Table 76 and Table 77 list characteristics of the warrants or guidelines included in the state manuals along with other procedures suggested in the literature. These tables also summarize the input data required, what is compared in the method, and the research team's observations and comments. Table 76 focuses on those procedures where additional evaluations were conducted as part of this research project. Findings from those evaluations are compared within this chapter. Table 77 provides information on other procedures mentioned in state manuals or in the literature.

Table 76. Characteristics of procedures evaluated.

Base Procedure	**Current Use**	**Input Data**	**How Evaluated**	**Comments**
Conflict Avoidance (Harmelink)	Several states use a derivation of the Harmelink procedure.	Approach volume per hour. Percent left turns. Opposing volume.	Minimum approach volume to avoid conflict is compared to actual approach volume.	Has long history of use. Updating of assumptions has been suggested by several authors. Methodology not appropriate for four-lane conditions.
Benefit-Cost Ratio	Not specifically referenced for left-turn treatments to date.	If site-specific data are required, could result in high level of input data. Using a procedure developed, say from this research project, would require volume, number of legs, and urban or rural determination.	Computes a benefit to cost ratio; left turn warranted if ratio is greater than 1.0 (or value determined by user). A B/C of 1.0 indicates benefits are greater than costs.	Considers delay savings, crash savings, and cost of treatment.
Crashes	Several state manuals note that crashes are to be considered.	Crashes.	Engineering judgment is typically used to decide if safety is a concern.	Development of turn-lane AMFs provides greater opportunity for additional consideration of crashes. Also available are equations to predict number of crashes, which requires volume, number of legs at the intersection, urban or rural, and, for rural, number of lanes per approach for the calculation. Can be a component of benefit-cost ratio.
Delay Reduction	A few states include criteria based on delay.	Volumes.	Volume curves are developed for comparison.	Procedure generally warrants left-turn lanes at higher volumes as compared to other approaches. Does not consider safety. Can be a component of benefit-cost ratio.
Cost	Mentioned in a few manuals.	Crash costs. Construction costs.	General guidance.	Can be a component of benefit-cost ratio.
Minimum Volume(s)	Some states include minimum volumes.	Number of left-turning vehicle.	Number of turning vehicles compared to threshold value.	Simple and easy to use.

Table 77. Characteristics of other procedures identified during the course of this study.

Base Procedure	Current Use	Input Data	How Evaluated	Comments
Capacity Evaluation	Capacity evaluations are suggested in several manuals.	Volumes. Roadway characteristics, etc.	Capacity evaluation conducted to determine needed treatments for desired level of service.	Capacity evaluations can include extensive efforts.
Speed Differential	Generally not mentioned or used.	Volumes. Perhaps arrival patterns.	Is anticipated speed difference greater than threshold value?	Need speed prediction for existing or anticipated conditions.
Policy/ Functional Classification	Considered in several state manuals.	Road characteristics (e.g., speed limit).	If characteristic is present, then left-turn lane is warranted.	
Design Consistency	Noted in several state manuals.	Design characteristics in the corridor.	General guidance.	
Sight Distance	Considered in several state manuals.	Speed.	Engineering analysis of available sight distance.	
Catalyst (Major, Spot, or Development)	Mentioned in a few state manuals.	Any of the above, plus reason left-turn lane is being considered.	Any of the above, plus consideration of reason left-turn lane is being considered.	Should different criteria be used for different funding sources?

CONFLICT AVOIDANCE (HARMELINK) PROCEDURE

The current AASHTO guideline for installing a left-turn lane at an unsignalized intersection is based on Harmelink's volume warrants. Harmelink's warrants are derived from queuing models considering the probability of a through vehicle arriving behind a stopped left-turn vehicle.

Limitations

A number of limitations of Harmelink's have been identified over the years. Following is a review of those identified limitations.

The same concept of limiting the probability of a through vehicle arriving behind a stopped left-turn vehicle is used for both two-lane and four-lane roadways in Harmelink's warrant development. Even though different values of maximum allowable probability are used for two-lane and four-lane roadways, this approach ignores the fact that lane changes can take place for through vehicles on the multilane approach to avoid being stopped behind a stopped left-turning vehicle. The queuing model-based probability may not directly correlate with the delay and safety impact experienced by the through vehicles on four-lane roadways. Therefore, revised Harmelink-based warrants were not generated for four-lane roadways as part of NCHRP Project 3-91.

Harmelink's warrants were developed based on allowable maximum probability of 0.020, 0.015, and 0.010 for operating speeds of 40, 50, and 60 mph, respectively, on two-lane highways. These values are subjective in nature and do not directly indicate stops, delay, speed change, or level-of-service degradation in any quantitative manner.

Harmelink assumed arrival time for both left-turn and through vehicles to be negative exponentially distributed, an assumption that is only valid for isolated intersections under low demand conditions. It has been reported by Kikuchi and Chakroborty (*19*) that there are problems in Harmelink's formulation. An inconsistency in the definitions of arrival and service rate in the queuing model could become a problem when through arrival demand becomes high. The total number of possibilities of making a left turn is overestimated because the formulation uses average gap values in the opposing flow that include unusable residual gaps.

Harmelink's warrants were developed based on dated model parameter values. Researchers have suggested that changes are needed for the time drivers use to complete a left turn, the time to clear the left-turn lane, and the critical gap. Harmelink's warrants are also based on "typical" drivers and geometry and cannot be used for special geometries and for safety evaluation of left turns at unsignalized intersections by special driver groups such as older drivers.

Comparison Between Harmelink Assumptions and Field Study Findings

The Harmelink procedure uses critical gap, time to clear the advancing lane, and time to clear the opposing lane(s), along with the probability of a through vehicle arriving behind a stopped, left-turning vehicle, to determine the warrants. Gap and clearance time values were gathered in the field studies conducted as part of this project (see Chapter 4). Table 78 compares the critical gap findings from the field studies to the values used by Harmelink. Table 79 shows the time-to-clear values from this field study and those assumed in the Harmelink procedure.

Table 78. Values for critical gap.

Conditions	Harmelink	Current Study
Crossing One Lane (11 to 12 ft)	5.0 sec	Raff/Hart: 5.12 sec Logit 50th: 4.60 sec Logit 85th: 6.26 sec
Crossing One Wide Lane (17 to 20 ft)		Raff/Hart: 5.43 sec Logit 50th: 5.14 sec Logit 85th: 6.62 sec
Crossing Either One or Two Lanes (20.5 to 23.5 ft)	6.0 sec	Raff/Hart: 5.35 sec Logit 50th: 4.34 sec Logit 85th: 6.51 sec
Crossing Two Lanes (24 to 27 ft)		Raff/Hart: 5.70 sec Logit 50th: 4.79 sec Logit 85th: 7.26 sec

Table 79. Values for time to clear.

Variable	Conditions	Harmelink	Average (Average + Standard Deviation) Values for:	
			Current Study (Subdivided by Number of Lanes)	Current Study (Grouped by Typical Crossing Width)
Clear Approach Lane, Average	Overall	1.9 sec, based on 150 vehicles	1.1 (1.6) sec, based on 2945 vehicles	
Clear Opposing Lane(s)	Two-lane highway or 11- to 12-ft crossing width	3.0 sec	2.5 (3.6) sec, based on 1181 vehicles	2.1 (2.6) sec, based on 277 vehicles and crossing width of 11 or 12 ft
	Four-lane highway or 22- to 27-ft crossing width	4.0 sec	2.7 (3.5) sec, based on 1764 vehicles	2.8 (3.6) sec, based on 1792 vehicles and crossing width of 22 to 27 ft
	Mixed	No value given	2.6 (3.4) sec, based on 2945 vehicles	2.4 (3.1) sec, based on 1792 vehicles and crossing width of 17 to 21 ft
Turning Time Equation Based on Field Study Findings	Overall	4.9 to 5.9 sec	$TT = 8.500324 + 0.017133 \times CW^2 - 0.511235 \times CW + 0.079616 \times \log(LGT) + 0.376136 \times \log(TAH) + 0.115794 \times \log(TIQ) - 0.045157 \times PSL - 0.525688 \times NOL2$ Where: TT = turning time (sec), LGT = accepted lag or gap time (sec), TAH = time at the head of the queue (sec), TIQ = time in the queue (sec), CW = crossing width (ft), CW_SQ = square of the crossing width (ft), PSL = posted speed limit (mph), and NOL2 = include indicator variable when number of lanes = 2.	

Critical Gap

Several recent research projects have determined the critical gap for use in intersection sight distance calculations and unsignalized intersection capacity analysis. As reported by Harwood et al. (*89*), Kyte et al. recommended a critical gap value of 4.2 sec for left turns from the major road by passenger cars for inclusion in the unsignalized intersection analysis procedures of the *Highway Capacity Manual* (*90*). A heavy-vehicle adjustment of 1.0 sec for two-lane highways and 2.0 sec for multilane highways was also recommended.

It is reasonable that design policies should be more conservative than operational criteria. Using a higher critical gap value accepted by 85 percent of the drivers, rather than the gap accepted by only 50 percent of the drivers, should result in a more conservative, design-oriented approach. With that philosophy, the authors of the 1996 intersection sight distance guidelines recommended a 5.5-sec gap value for use in intersection sight distance (*89*). This gap value is to be increased to 6.5 sec for single-unit trucks and 7.5 sec for combination trucks. Also, an additional 0.5 sec for cars and 0.7 sec for trucks should be added when crossing an additional opposing lane.

Harmelink's assumptions of 5.0 sec and 6.0 for critical gap on two-lane and four-lane highways, respectively, are less than or similar to the average values identified in this research. If a more conservative gap value for use in design is desired, then the critical gap value should be increased to the following:

- 6.25 to 6.50 sec for two-lane highways and
- 6.50 to 7.25 sec for four-lane highways.

Time to Clear

This research project found differences in the time to clear the approach lane and the time to clear the opposing lane as compared to Harmelink's assumptions (see Table 79). Both the average time to clear the approach lane and the average time to clear the opposing lane were smaller (or faster). The average plus standard deviation value was also smaller than Harmelink's assumptions in most situations. The only condition when the average plus standard deviation value was larger was for the two-lane highway sites when using the data for all two-lane highway sites (3.6 sec as compared to 3.0 sec).

Changes to Warrants

The left-turn lane warrants would shift downward (i.e., left-turn lanes would be warranted at lower volumes) for two-lane highways if the following are assumed:

- Critical gap of 6.25 sec,
- Time to clear the advancing lane of 1.6 sec,
- Time to clear the opposing lane of 3.6 sec, and
- No change to the assumed probability of a through vehicle arriving behind a stopped left-turning vehicle.

Figure 38 illustrates the change from the *Green Book* warrants to the warrants that would be generated based on the field study findings for a 60-mph two-lane highway.

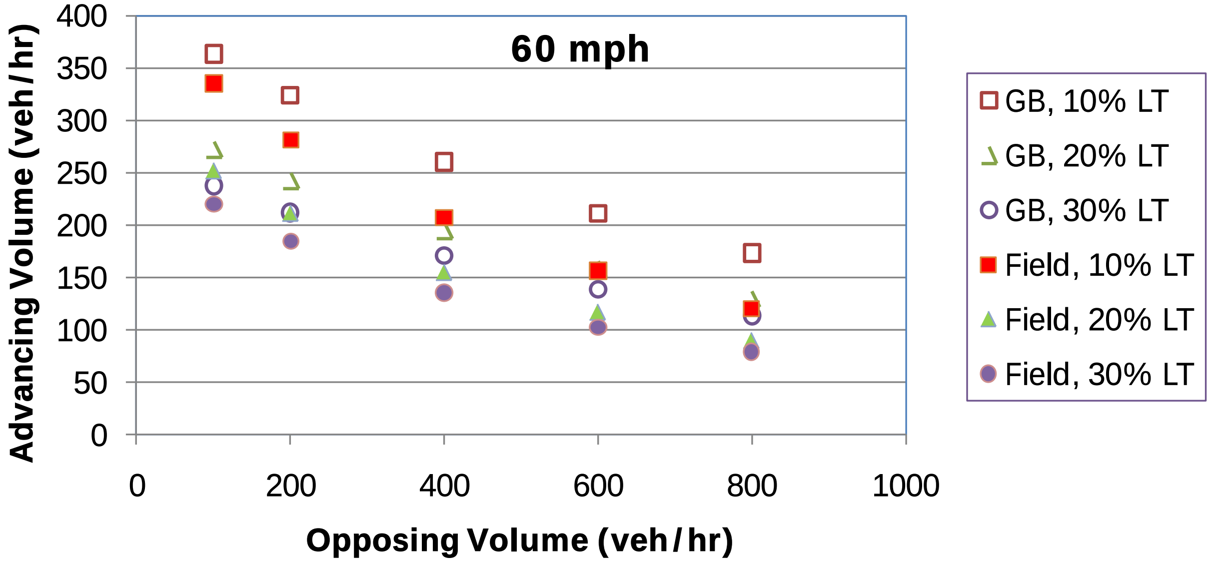


Figure 38. Example of change in *Green Book* (GB) left-turn lane warrants if findings from field study (Field) are used in the Harmelink procedure for a 60-mph two-lane highway.

BENEFIT-COST RATIO

Economic criteria for left-turn lanes were established by determining the level of peak-hour major-road volume and peak-hour conflicting left-turn volume that result in a B/C ratio equal to 1.0 and 2.0. A range of crash costs and a range of construction costs were evaluated. The research team recommends that the mid-range crash cost and the moderate construction cost identified as part of this research be considered in developing the final left-turn lane warrant recommendations. A benefit-cost range of 1.0 to 2.0 can also provide consideration of potential variability in the assumptions. A benefit-cost of 1.0 represents the point when the calculations find benefits outweigh costs. Using a ratio of 2.0 may represent a more practical application of the benefit-cost evaluation.

The left-turn lane warrants developed using the moderate construction cost and mid-range crash cost along with a benefit-cost of 1.0 and 2.0 are listed in Table 73 and also shown in Figure 35 and Figure 36.

For rural two-lane highways, a left-turn lane is warranted for as low as five left-turning vehicles in 1 hour when opposed by 50 major-road vehicles per hour. For rural four-lane highways, a left-turn lane is warranted when five left-turn vehicles oppose 50 veh/hr/ln for a four-leg intersection or 75 veh/hr/ln for a three-leg intersection. For urban four-leg intersections, a left-turn lane is to be considered when turning across 50 veh/hr/ln in the peak hour. Urban three-leg intersections had the biggest range of warrant recommendations as shown in Figure 35.

The suggested warrants are:

- Use B/C = 1.0 for urban and suburban arterials (see Table 80 and Figure 39).
- Use B/C = 1.0 for rural two-lane highways to warrant a bypass lane (see Table 81 and Figure 40).

- Use the results based on B/C = 2.0 for rural two-lane highways to warrant a left-turn lane (see Table 81 and Figure 40).
- Use B/C = 1.0 for rural four-lane highways to warrant a left-turn lane (see Table 82 and Figure 41).

Table 80. Suggested left-turn lane warrants based on results from benefit-cost evaluations for urban and suburban arterials.

Left-Turn Lane Peak-Hour Volume (veh/hr)	Three-Leg Intersection, Major Urban and Suburban Arterial Volume (veh/hr/ln) That Warrants a Left-Turn Lane	Four-Leg Intersection, Major Urban and Suburban Arterial Volume (veh/hr/ln) That Warrants a Left-Turn Lane
5	450	50
10	300	50
15	250	50
20	200	50
25	200	50
30	150	50
35	150	50
40	150	50
45	150	< 50
50 or More	100	< 50

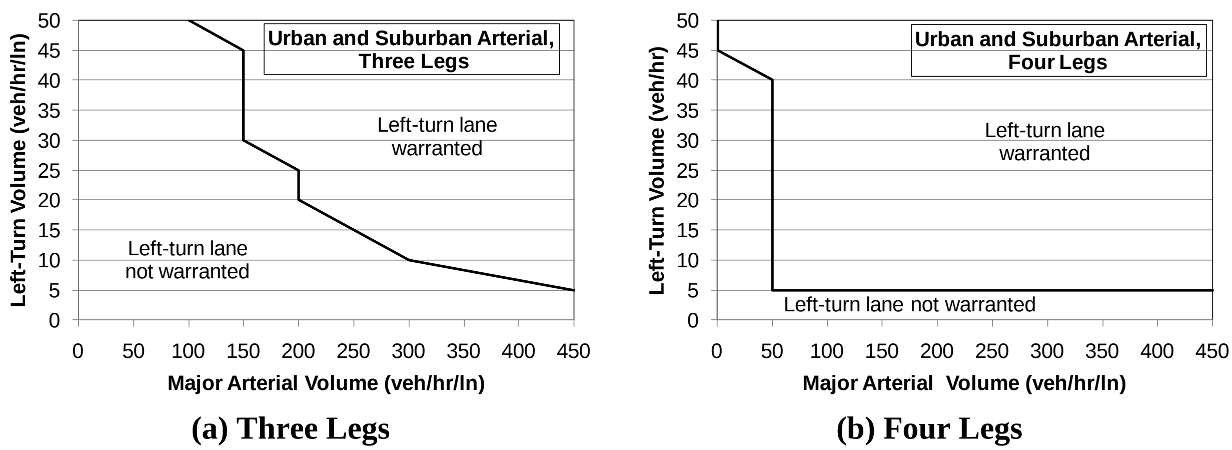
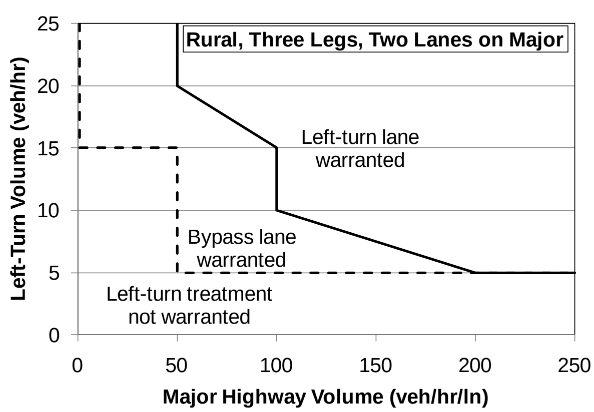


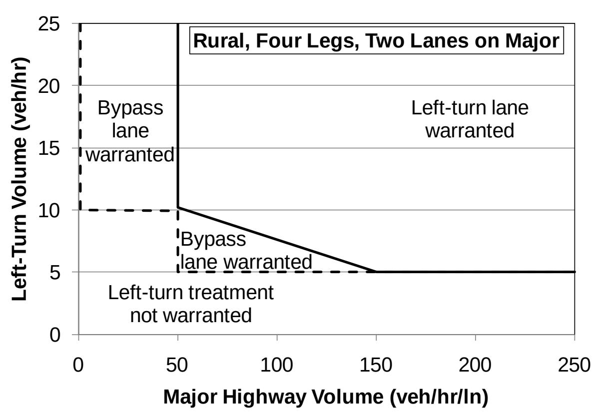
Figure 39. Suggested left-turn lane warrants based on results from benefit-cost evaluations for intersections on urban and suburban arterials.

Table 81. Suggested left-turn treatment warrants based on results from benefit-cost evaluations for rural two-lane highways.

Left-Turn Lane Peak-Hour Volume (veh/hr)	Three-Leg Intersection, Major Two-Lane Highway Peak-Hour Volume (veh/hr/ln) That Warrants a Bypass Lane	Three-Leg Intersection, Major Two-Lane Highway Peak-Hour Volume (veh/hr/ln) That Warrants a Left-Turn Lane	Four-Leg Intersection, Major Two-Lane Highway Peak-Hour Volume (veh/hr/ln) That Warrants a Bypass Lane	Four-Leg Intersection, Major Two-Lane Highway Peak-Hour Volume (veh/hr/ln) That Warrants a Left-Turn Lane
5	50	200	50	150
10	50	100	< 50	50
15	< 50	100	< 50	50
20	< 50	50	< 50	< 50
25	< 50	50	< 50	< 50
30	< 50	50	< 50	< 50
35	< 50	50	< 50	< 50
40	< 50	50	< 50	< 50
45	< 50	50	< 50	< 50
50 or More	< 50	50	< 50	< 50



(a) Three Legs

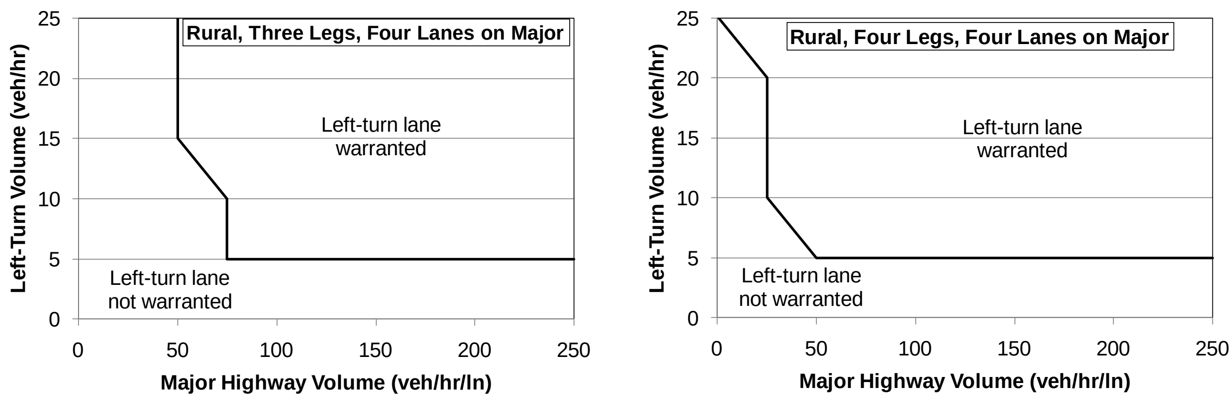


(b) Four Legs

Figure 40. Suggested left-turn treatment warrants based on results from benefit-cost evaluations for intersections on rural two-lane highways.

Table 82. Suggested left-turn lane warrants based on results from benefit-cost evaluations for rural four-lane highways.

Left-Turn Lane Peak-Hour Volume (veh/hr)	Three-Leg Intersection, Major Four-Lane Highway Peak-Hour Volume (veh/hr/ln) That Warrants a Left-Turn Lane	Four-Leg Intersection, Major Four-Lane Highway Peak-Hour Volume (veh/hr/ln) That Warrants a Left-Turn Lane
5	75	50
10	75	25
15	50	25
20	50	25
25	50	< 25
30	50	< 25
35	50	< 25
40	50	< 25
45	50	< 25
50 or More	50	< 25



(a) Three Legs **(b) Four Legs**

Figure 41. Suggested left-turn lane warrants based on results from benefit-cost evaluations for intersections on rural four-lane highways.

Figure 42 shows the suggested LTL installation guidelines from this research as compared to the guidelines from the departments of transportation (DOTs) of Georgia (GA), New Mexico (NM), and South Dakota (SD). Figure 43 is a similar graph that shows the suggested left-turn bypass lane guidelines from this research as compared to the guidelines from the Georgia DOT. These guidelines are for rural two-lane highways with speeds of 55 mph or greater. Locations on the figure that are above and/or to the right of a particular threshold are conditions in which a left-turn lane, bypass lane, or deceleration lane are warranted.

The figures show that the suggested guidelines from this research provide for the installation of a left-turn lane for lower volumes of opposing traffic than the DOT guidelines in most cases. New Mexico has the lowest left-turn volume threshold (5 veh/hr) among the states, for the installation of a deceleration lane; this threshold largely agrees with that of Georgia for a right-hand bypass lane. For combinations of very low opposing volumes and turning volumes, these two states have more permissive guidelines than those developed in this research; for other conditions, this research provides the lowest installation thresholds. Comparisons of guidelines for rural four-lane highways and for suburban/urban arterials produced similar results.

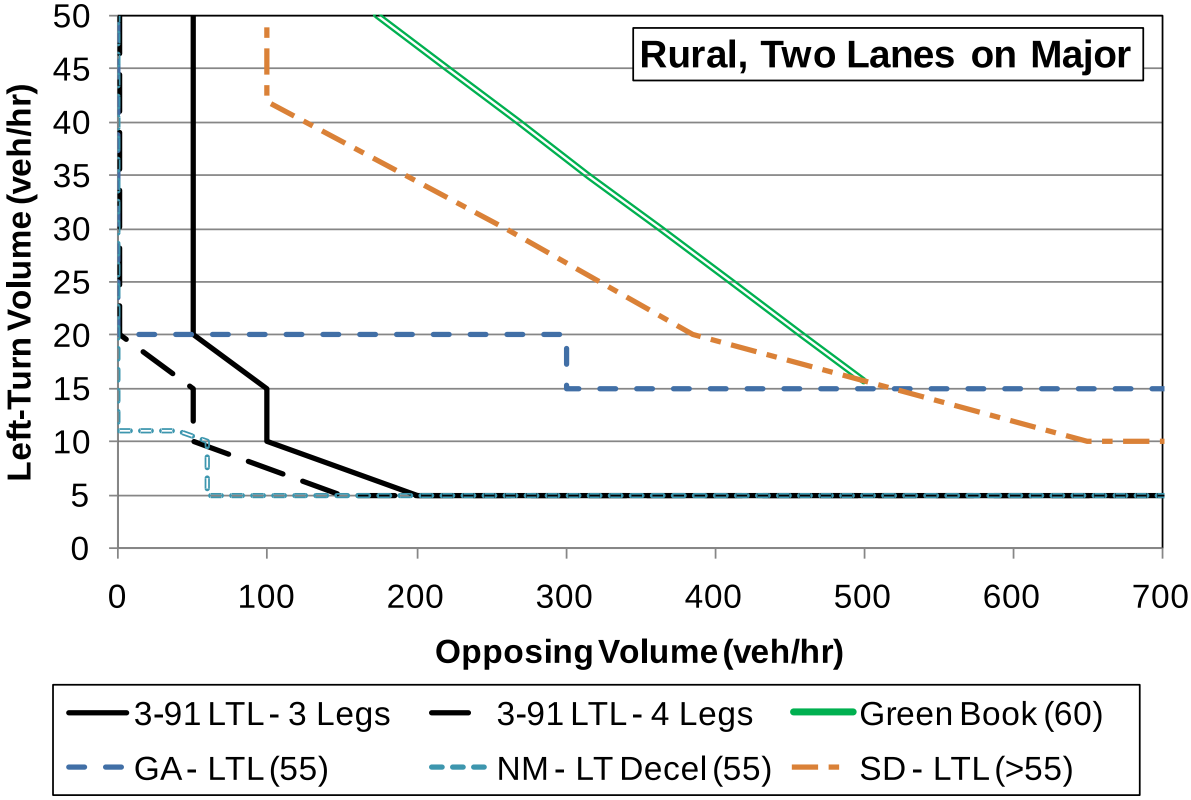


Figure 42. Comparison of suggested left-turn lane warrants for rural two-lane highways.

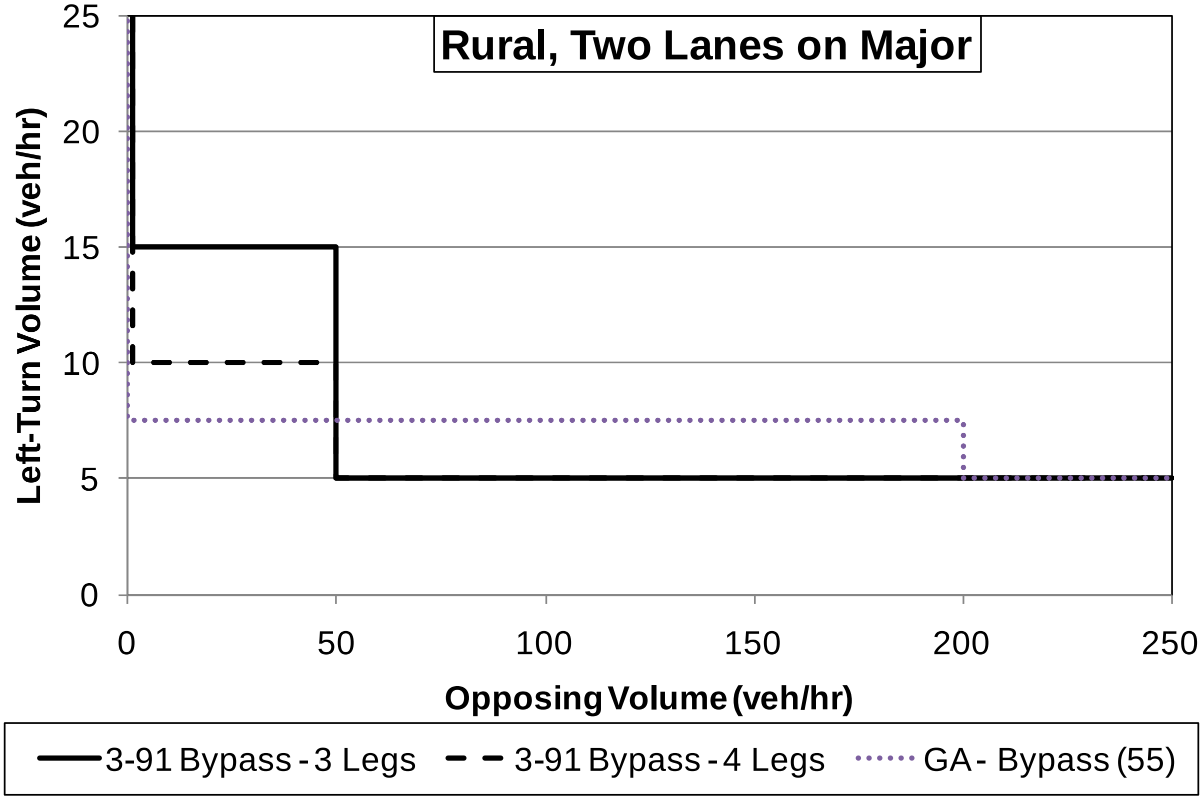


Figure 43. Comparison of suggested bypass lane warrants for rural two-lane highways.

MINIMUM VOLUME/ENGINEERING JUDGMENT

Minimum volume criteria are considered in different criteria recommended by researchers and used in some states. For example, *NCHRP Report 348* criteria by Koepke and Levinson provide two methods for determining the need for left-turn lanes (*15*). The first method is shown in Figure 6; a left-turn lane is warranted if more than 30 vehicles are turning left for 30- to 35-mph roadways (25 vehicles for 40- to 45-mph roadways). However, Koepke and Levinson state that in most cases, left-turn lanes should be provided where there are more than 12 left turns per peak hour. The benefit-cost ratio identified scenarios for rural highways when a left-turn lane is justified with as few as 5 veh/hr turning left when turning across as few as 50 veh/hr/ln. The research team recommends that the values in Table 80 and Table 81 be used rather than a minimum volume.

The characteristics of the field study sites along with a selection of the case study sites included in the *Design Guide* (*4*) were used in the benefit-cost and Harmelink procedures. The results were reviewed to identify whether they generated logical and appropriate results. In the engineering judgment of the research team, the results from the benefit-cost approach are reasonable.

LEFT-TURN LANE STORAGE

The left-turn lane should be sufficiently long to store the number of vehicles likely to accumulate during a critical period; the definition of that critical period can vary depending on the traffic conditions at the site. Regardless of the specific critical period, the storage length should be sufficient to avoid the possibility of the left-turning queue spilling over into the through lane.

According to the *Green Book* (*5*), at unsignalized intersections, the storage length, exclusive of taper, may be based on the number of turning vehicles likely to arrive in an average 2-minute period within the peak hour. Space for at least two passenger cars should be provided; with over 10 percent truck traffic, provisions should be made for at least one car and one truck. Table 83 shows the recommended spacing by percent truck included in the TRB *Access Management Manual* (*91*).

Table 83. Queue storage length per vehicle (*91*).

Trucks (Percent)	Assumed Queue Storage Length (ft) per Vehicle in Queue
≤ 5	25
10	30
15	35

The 2-minute waiting time suggested in the *Green Book* may need to be changed to some other interval that depends largely on the opportunities for completing the left-turn maneuver. These intervals, in turn, depend on the volume of opposing traffic, which the *Green Book* does not address. For additional information on storage length, the *Green Book* refers the reader to the *Highway Capacity Manual* (*92*). The first equation shown in Table 84 can be used to determine the design length for left-turn storage as described by the *Green Book.*

Many states use the *Green Book* method, a method based on work done by Harmelink (*1*) (see Figure 44), or a method based on work by Jack E. Leisch and Associates (*93*) (see Figure 45) to describe their recommended storage lengths in their design guidelines. Others recommend that the designer assume that the intersection is signalized with a two-phase signal using a 40- to 60-sec cycle length, and then use the *Highway Capacity Manual* methodology to determine the expected storage length. The authors of *NCHRP Report 348* (*15*) state that the required storage length of a left-turn lane depends upon the likely left-turn volumes during the peak 15 minutes of the design hour, which is typically but not always the morning or evening peak hour. The length for a stop-controlled lane should be adequate 95 percent of the time and can be estimated by using the cumulative Poisson distribution.

Table 84. Equations used to determine storage length.

Equation in TRB *Access Management Manual*

$$L = \frac{V}{N_c} ks \qquad (72)$$

Where:

- L = design length for left-turn storage (ft);
- V = estimated left-turn volume, vehicles per hour (veh/hr);
- N_c = number of cycles per hour (for the *Green Book* unsignalized procedure, this would be 30 [V/N is the average number of turning vehicles per cycle]);
- k = factor that is the length of the longest queue (design queue length) divided by the average queue length (a value of 2.0 is commonly used for major arterials, and a value of 1.5 to 1.8 might be considered for an approach on a minor street or on a collector where capacity will not be critical) (for the *Green Book* procedure this would be 1.0); and
- s = average length per vehicle, including the space between vehicles, generally assumed to be 25 ft (adjustments are available in several documents for trucks and buses, such as the TRB *Access Management Manual* [see Table 83]).

Equations Used in *NCHRP Report 457*

Equations also used to generate values in Table 85

$$P(n > N) = \left(\frac{v}{c}\right)^{(N+1)} \qquad (73)$$

$$c = \frac{V_o e^{-V_o t_c/3600}}{1 - e^{-V_o t_f/3600}} \qquad (74)$$

$$N = \frac{ln[P(n>N)]}{ln[v/c]} - 1 \qquad (75)$$

$$SL = N \times VL = \left\{\frac{ln[P(n>N)]}{ln[v/c]} - 1\right\} \times VL \qquad (76)$$

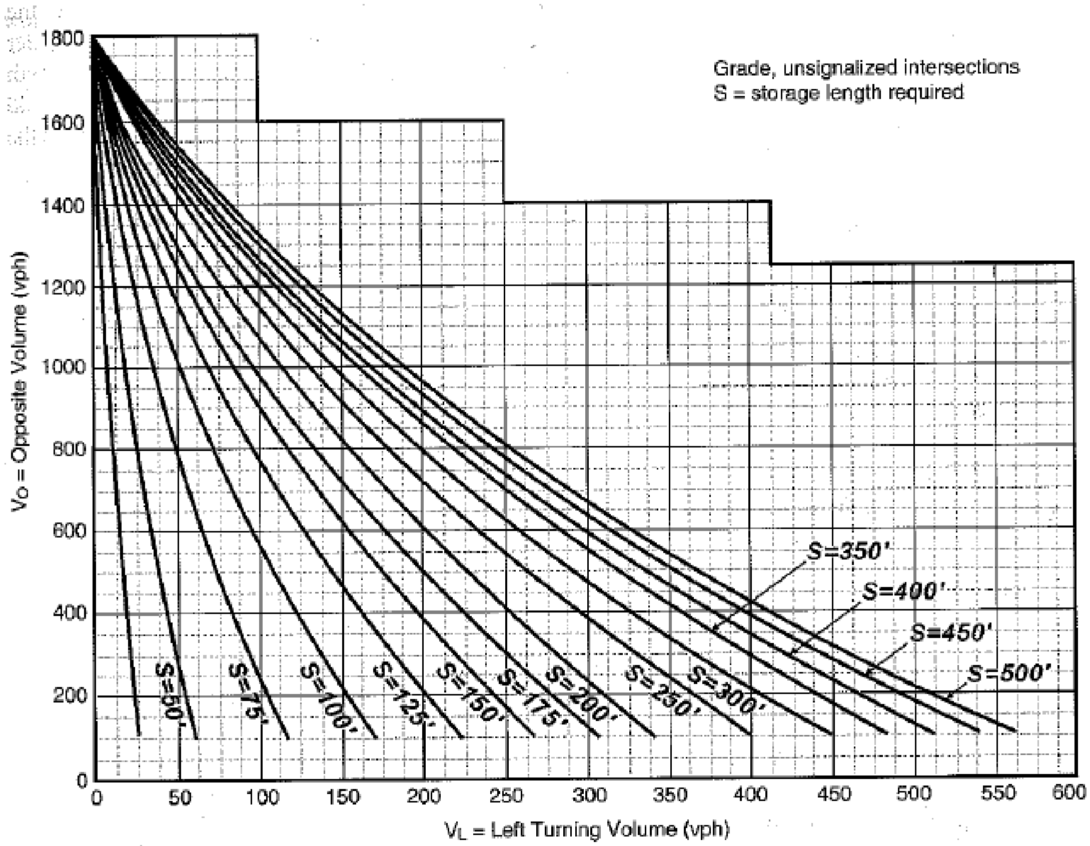
Where:

- $P(n>N)$ = probability of bay overflow;
- v = left-turn vehicle volume (veh/hr);
- N = number of vehicle storage positions;
- c = movement capacity (veh/hr);
- V_o = major-road volume conflicting with the minor movement, assumed to be equal to one-half of the two-way major-road volume (veh/hr);
- SL = storage length (ft);
- t_c = critical gap (sec);
- t_f = follow-up gap (sec); and
- VL = average length per vehicle, including the space between vehicles, generally assumed to be 25 ft (adjustments are available in several documents for trucks and buses such as the TRB *Access Management Manual* [see Table 83]).

Table 85. Recommended storage lengths from *Access Management Manual* equation and *NCHRP Report 457* equations with revised critical gap.

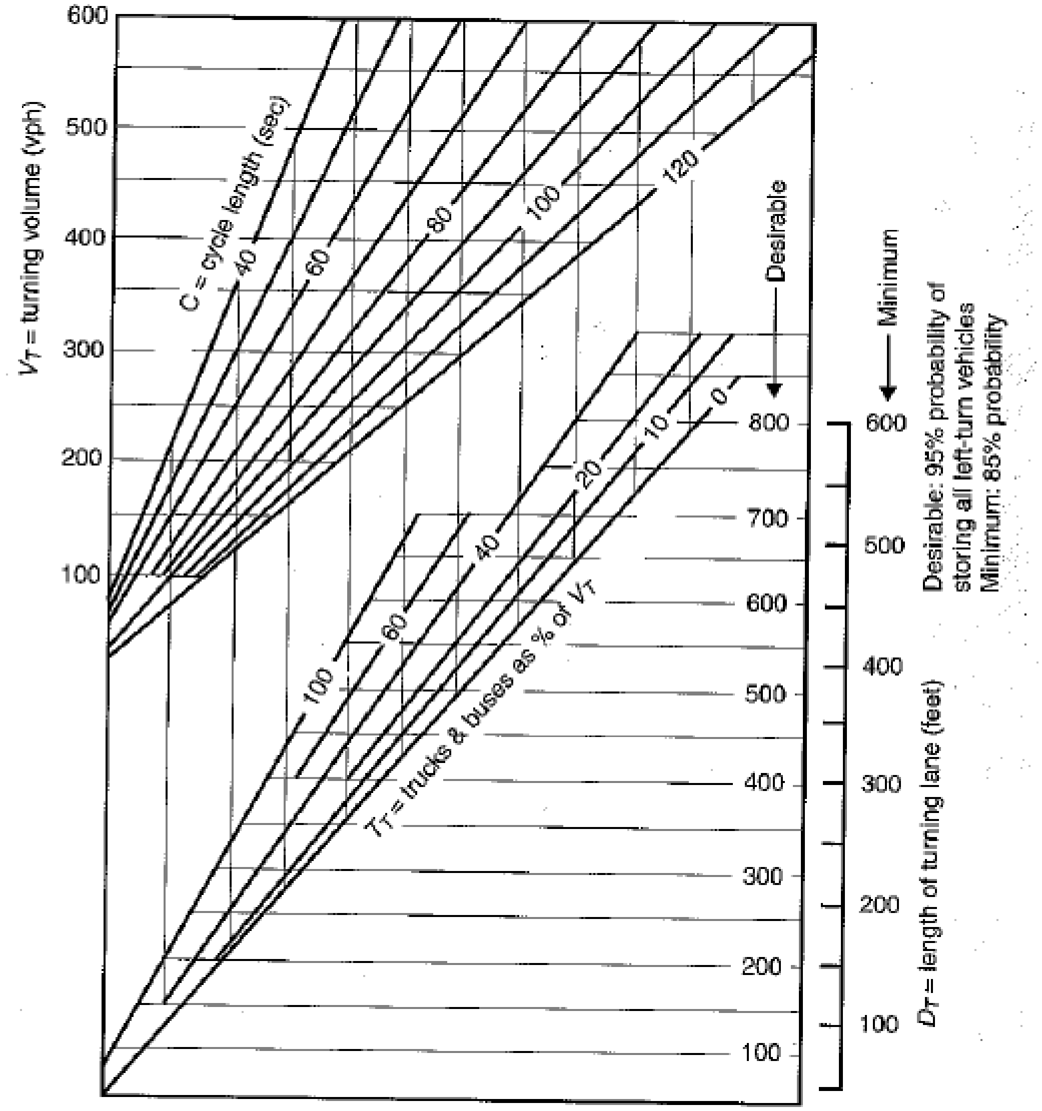
Left-Turn Volume (veh/hr)	Storage Length, Rounded Up to Nearest 25-ft Increment (ft)						
	Storage Lengths from Other Manuals for Comparison		Storage Lengths Calculated from Equations [b] Documented in *NCHRP Report 457* Using Revised Critical Gaps and 0.005 Probability of Overflow				
	Green Book Procedure ($k = 1$)[a]	Equation ($k = 2$)[a]	Opposing Volume (veh/hr)				
			200	400	600	800	1000
Critical Gap = 5.0 sec, Follow-Up Gap = 2.2 sec (Represents the 50th Percentile Critical Gap Found in Field Studies)							
40	75	75	50	50	50	50	50
60	50	100	50	50	50	50	50
80	75	150	50	50	50	50	50
100	100	175	50	50	50	50	75
120	100	200	50	50	50	75	75
140	125	250	50	50	50	75	75
160	150	275	50	50	75	75	100
180	150	300	50	50	75	75	100
200	175	350	50	75	75	100	125
220	200	375	50	75	75	100	125
240	200	400	75	75	100	125	150
260	225	450	75	75	100	125	175
280	250	475	75	75	100	125	175
300	250	500	75	100	125	150	200
Critical Gap = 6.25 sec, Follow-Up Gap = 2.2 sec (Represents the 85th Percentile Critical Gap Found in Field Studies, 85th Percentile Is Preferred for Design)							
40	75	75	50	50	50	50	50
60	50	100	50	50	50	50	50
80	75	150	50	50	50	50	75
100	100	175	50	50	50	75	75
120	100	200	50	50	75	75	100
140	125	250	50	50	75	100	125
160	150	275	50	75	75	100	150
180	150	300	50	75	75	125	150
200	175	350	50	75	100	125	200
220	200	375	75	75	100	150	225
240	200	400	75	75	125	150	275
260	225	450	75	100	125	175	325
280	250	475	75	100	125	200	400
300	250	500	75	100	150	225	525

[a, b] See Table 84 for equations.
This table assumes 25 ft per vehicle spacing. Table 83 provides other suggested spacing lengths based on percent trucks.



Source: Harmelink, M., "Volume Warrants for Left-Turn Storage Lanes at Unsignalized Grade Intersections," in *Highway Research Record 211*, Figure 1, p. 8. Copyright, National Academy of Sciences, Washington, D.C., 1967. Reproduced with permission of the Transportation Research Board.

Figure 44. Storage length recommendations based on work by Harmelink (*1*).



Example

Conditions	Solution
• Left-turn volume, V_T = 240 vph • 120-second cycle length • No trucks or buses	• Desirable storage = 400 ft • Minimum storage = 300 ft

Source: TRB Committee on Access Management, *Access Management Manual*, Figure 10-7, p. 174. Copyright, National Academy of Sciences, Washington, D.C., 2003. Reproduced with permission of the Transportation Research Board.

Figure 45. Storage length recommendations based on work by Leisch (*93*) as presented in the TRB *Access Management Manual* (*91*).

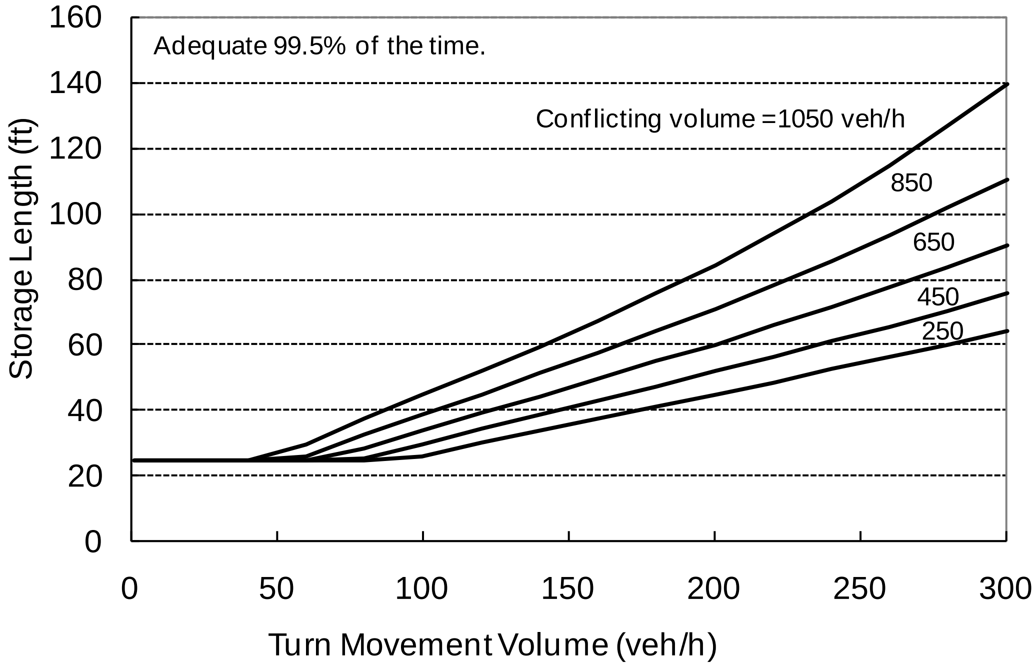
NCHRP Report 457 (*12*) developed suggested storage length values using the equations shown in Table 84. The *NCHRP Report 457* procedure was similar to Harmelink's work regarding storage length of left-turn bays at unsignalized intersections. The storage length equation is a function of movement capacity, which is dependent upon assumed critical gap and follow-up gap. Critical gap is defined by the *Highway Capacity Manual* as the minimum time interval in the major-street traffic stream that allows intersection entry for one minor-street vehicle. Thus, the driver's critical gap is the minimum gap that would be acceptable. The time between the departure of one vehicle from the minor street and the departure of the next vehicle using the same major-street gap, under a condition of continuous queuing on the minor street, is called the follow-up time.

NCHRP Report 457 used a smaller critical gap (4.1 sec as recommended in the *Highway Capacity Manual* compared to the 5.0 or 6.0 sec used by Harmelink for two-lane and four-lane highways, respectively), which resulted in shorter values than those generated by Harmelink. The follow-up gap was assumed to be 2.2 sec as recommended in the *Highway Capacity Manual*. The assumptions made regarding critical gap and the resulting capacity for the movement used in these procedures can have a significant effect on the calculated storage length recommendations as demonstrated by several researchers (*94*, *62*, *12*).

It is generally recognized that a storage area should adequately store the turn demand a large percentage of the time (e.g., 95 percent or more, which means that the demand would exceed the storage length less than or equal to 5 percent of the time). A 0.5 percent limit was used for the major-road left-turn bay lengths in *NCHRP Report 457* based on the recommendation of Harmelink. This smaller limit reflects the greater potential for severe consequences when a bay overflows on an unstopped, major-road approach. The critical and follow-up gaps were assumed to equal 4.1 and 2.2 sec, respectively. Figure 46 shows the graphical representation of the storage length guidelines presented in *NCHRP Report 457*; the Internet version of that document also provides an interactive spreadsheet tool in which the designer can input the specific volume and gap variables to receive a recommended storage length for those conditions. *NCHRP Report 457* assumed a 25-ft minimum storage length.

Harmelink used larger values for critical gap (5.0 sec for two-lane highways and 6.0 sec for four-lane highways). When those gaps are used within the approach presented by Bonneson and Fontaine in *NCHRP Report 457*, storage lengths similar to those suggested by Harmelink are obtained. When the critical gap of 5.0 and 6.25 sec determined in NCHRP Project 3-91's field studies are used, the storage lengths shown in Table 85 are generated.

Each of the sources emphasizes that the appropriate storage length is dependent on both the volume of turning traffic and the volume of opposing traffic. If volume data are not available, for urban and suburban streets with lower speeds (e.g., less than 40 mph), it is recommended that the minimum storage length be at least 50 ft to accommodate two cars; for high-speed and rural locations, a minimum storage length of 100 ft is recommended.



Source: Bonneson, J., and M. Fontaine, *Engineering Study Guide for Evaluating Intersection Improvements*, *NCHRP Report 457,* Figure 2-7, p. 24. Copyright, National Academy of Sciences, Washington, D.C., 2001. Reproduced with permission of the author.

Figure 46. Recommended storage lengths for left-turn lanes at uncontrolled approaches using a 25-ft minimum storage length along with a critical gap of 4.1 sec *(12*).

CHAPTER 7

SUMMARY AND CONCLUSIONS

SUMMARY

Left-turn movements at intersections, including driveways—especially movements that are made from lanes that are shared with through traffic—cause delays and adversely impact safety. The left-turn problem has been given relatively little attention at unsignalized intersections. Although updated warrants have been developed in some jurisdictions for when to provide left-turn lanes, many agencies still use research from the mid-1960s. Current conditions require a broader assessment of when to provide left-turn accommodations. Technical warrants are an important element of the decision-making process.

Objective

The key objectives for this NCHRP project were to:

- Develop an objective and clear process for the selection of left-turn accommodations at unsignalized intersections and
- Provide guidance on the design of these accommodations.

The findings presented in the sections below provide the recommended warrants for left-turn lanes based on the research conducted in this project. A companion document, the *Design Guide on Left-Turn Accommodations at Unsignalized Intersections* (*4*), provides guidance on the design of left-turn lanes. Other objectives for the project included:

- Review left-turn lane installation guidelines available in the literature or state manuals,
- Review the design guidance provided in the literature or state manuals,
- Conduct a legal review,
- Conduct interviews to determine the state of the practice regarding left-turn lane installations at unsignalized intersections,
- Compare different methods used or suggested for determining when to install a left-turn lane,
- Update the assumptions used in the Harmelink procedure, and
- Generate warrants based on an economic procedure that considers both delay savings and benefits resulting from a reduction in crashes.

Literature Review

A review of the literature was performed using many sources, including research reports, state and federal design manuals, and handbooks. Although many procedures are currently in use by various organizations to determine the need for left-turn lanes, several are either very similar or identical. Most states' criteria are based on the *Green Book* values or the *NCHRP Report 279* values—both of which are based on M. Harmelink's work.

Legal Review

The legal review addressed the following question: When a government seeks to fulfill a broad public objective such as safety and, in this project, left-turn accommodation, who should bear the costs—the developer who would be adding traffic to the roadway network or the general public? Unless the facts of the case are clear cut, the outcome is unpredictable even within specific jurisdictions. Results will be more predictable if agencies have the documented authority to manage access. This authority could be achieved in a number of different ways (state or municipal codes for access management, administrative rules, etc.). Otherwise, there are too many hazy cases and multiple factors so that predicting the outcome is impossible.

Interviews

To help investigate the implementation of left-turn accommodations at unsignalized intersections, researchers conducted interviews of representatives from state DOTs, county governments, city governments, and consultants. The 25 questions in the interview were structured into planning, design, legal/policy/finance, and potential future applications.

All interview participants indicated that left-turn treatments are provided at unsignalized intersections. In addition to left-turn lanes, all state DOTs, all cities, and a majority of the counties in the survey indicated they use two-way, left-turn lanes. Several participants noted that TWLTLs are typically used in areas where there may be poor access control. Several state DOTs and two counties indicated they consider bypass or shoulder widening, but they noted that these techniques are typically used when there are limited options. One state DOT noted that roundabouts are another treatment considered for dealing with left-turn movements.

The state DOT process for determining where left-turn treatments should be installed at unsignalized locations, who is involved, and what affects the decision may vary depending upon the circumstances involved and whether the question relates to:

- A developer seeking access onto the roadway system for a new development or major redevelopment,
- The state DOT reevaluating the roadway condition as part of an improvement project,
- A county- or city-initiated project that is being coordinated with the state DOT, or
- A problem location identified by citizen complaints or its crash rate.

Two of the state DOTs reflected in the survey had two interview participants. Based on responses from these two state DOTs, it appears practices may vary depending on the area of the state or which DOT unit is involved.

A majority of the interview participants at all levels were not aware of the two Supreme Court decisions relating to essential nexus or rough proportionality—*Nollan v. California Coastal Commission* and *Dolan v. City of Tigard*. Five state DOT and two county respondents were aware of the Supreme Court decisions. All of the interview participants who expressed an awareness of the Supreme Court decisions indicated they were aware of no ramifications of these court cases to their jurisdictions' decisions related to left-turn accommodations. Those participants indicated that their policies are consistent with these decisions and already reflected rough proportionality, rational nexus, etc.

Comparison of Existing Procedures

The use of the Harmelink criteria requires approaching volume, opposing volume, left-turn percentage, and speed of the roadway. Those states that have adopted other criteria generally have fewer required input values; for example, one state only requires the anticipated left-turn volume, one state only requires the anticipated minor-road volume, and another state has criteria based on ADT and left-turning volume by posted speed. In addition to having fewer demands for input values, the newer procedures also result in left-turn lanes being warranted at lower volumes than the criteria currently in the *Green Book.*

While most states use procedures that are based on Harmelink, a number of limitations of Harmelink's procedure have been identified over the years. Harmelink's warrants are developed based on allowable maximum probability values that are subjective in nature and do not directly indicate stops, delay, speed change, or level-of-service degradation in any quantitative manner. Harmelink assumed arrival time for both left-turn and through vehicles to be negative exponentially distributed, an assumption valid only for isolated intersections under low demand conditions.

Driver Behavior Study

This project's driver behavior study used videotaped recordings of vehicle and pedestrian operations to obtain needed data for calibrating the simulation model and updating the Harmelink procedure. Left-turn movements were studied at 30 sites that were located in College Station/Bryan, Texas; Houston, Texas; Staten Island, New York; and Phoenix, Arizona. The sites were selected based on a variety of intersection arrangements and geometric characteristics, including:

- Number of lanes on the major road—two or four lanes;
- Presence of a left-turn lane—yes or no;
- Signal coordination—location is near enough to a signal to be affected or far enough from a signal to result in random arrival; and
- Approach speed range—low or high speed, with posted speed limits between 25 and 40 mph being defined as low speed and posted speed limits of 45 mph or more being defined as high speed.

The reduction process began with reviewing the site video and obtaining turning movement counts using 5-minute intervals. The goal for each site was to obtain data for a minimum of 100 left-turning vehicles whose drivers had to make a decision based on the available gaps in the opposing traffic. The behaviors for a total of 3570 vehicles were collected from the field studies. Heavy trucks only represented a small portion of the data collected (39 vehicles), and since their operations are known to be slower than passenger cars, they were excluded from the evaluations. Of the remaining vehicles, 2945 vehicles started from a stopped position. A vehicle was included as starting from a stopped position if the vehicle spent at least 0.25 sec between arriving at the front of the queue and starting the left-turn maneuver.

The most influential variables on the amount of time used to clear the intersection are crossing width and posted speed limit. As an example, a change from a crossing width of 11 ft to 27 ft resulted in an additional 2.24 sec in total turning time. The posted speed limit variable is associated with a decrease in turning time for the higher speeds. For turning time, an additional 1.81 sec is subtracted when the roadway has a 65-mph posted speed as compared to a 25-mph posted speed. The posted speed limit range represented in the dataset was 25 to 65 mph.
The relationship between clearance time and the accepted lag or gap time was in the direction expected although it did not have as large an influence as initially thought. The preliminary thought was that drivers will notably drive faster (i.e., lower turning time) when accepting a small gap. The evaluation only found a small increase in clearance time for larger gaps. For a 20-sec gap as compared to a 5-sec gap, the increase in turning time was about 0.05 sec.

Critical gap is defined as the time interval between two opposing vehicles that is necessary for a left-turning vehicle to safely complete a left-turn maneuver. Two methods were used to determine critical gap: logistic regression and Raff/Hart. Logistic regression is appropriate when the dependent variable is binary or dichotomous (e.g., either the acceptance or rejection of a gap). The relationship between posted speed limits from the field studies was similar to the finding from a simulator study reported in the literature—smaller gaps are accepted at higher posted speed sites. The difference is on the order of 1 to 1.5 sec. Gap acceptance values increase as the crossing width increases, although only by a small amount (less than 1 sec between the one-lane group and the two-lane or very-wide-one-lane group).

Updating Harmelink

The findings from the field studies were used to update Harmelink assumptions for two-lane rural highways. The time-to-clear values were smaller at the 30 field study sites, and the critical gap was larger, resulting in a shift of the left-turn warrants downward. In other words, left-turn lanes would be warranted at lower volumes. Because of several concerns with the Harmelink procedure, including the lack of a clear relationship between the assumptions in the model and delay or safety on a highway, the research team recommends that the results from the benefit-cost ratio be used as the basis for left-turn lane warrants.

Updating Harmelink Storage Lengths

Harmelink also provided left-turn lane storage lengths in his research. Storage lengths were developed in this project using the method presented in *NCHRP Report 457* along with critical gap values found in this field study (5.0 sec and 6.25 sec, representing 50th and 85th percentiles, respectively). These lengths were compared to lengths generated using other methods, such as the method discussed in the *Green Book.*

Economic Analysis for Existing Sites

Economic analysis can provide a useful method for combining traffic operations and safety benefits of left-turn lanes to identify situations in which left-turn lanes are and are not justified economically. A benefit-cost approach was used in this project to determine when a left-turn lane would be justified. The steps included simulation to determine delay savings from installing a left-turn lane, crash costs, crash reduction savings determined from safety performance functions and accident modification factors available in the *Highway Safety Manual*, and construction costs. The comparison identifies the benefits when the left turns at an existing driveway or intersection are provided a left-turn lane.

The total average delay for when a left-turn lane is present is subtracted from the total average delay when a left-turn lane is not present. This difference represents the total average delay savings per vehicle at the intersection on the major roadway. The estimated reduction in delay provided by a left-turn lane on a two-lane highway ranged from 0 to 4 sec/veh when the major-road volume ranged from 400 to 800 veh/hr/ln. The delay reduction for four-lane highways generally ranged between 0 and 2 sec/veh for volumes up to 800 veh/hr/ln on the major road and left-turning vehicle volume of 100 veh/hr or less. Delays were much higher when the left-turning volume was 140 veh/hr or greater and the major-road volume was 600 veh/hr/ln or greater.

The predicted average crash frequency for an intersection can be determined from equations in the *Highway Safety Manual*. These equations, called safety performance functions, are regression models for estimating the predicted average crash frequency of individual roadway segments or intersections for a set of specific base conditions. As discussed in the HSM, each SPF in the predictive method was developed with observed crash data for a set of similar sites. The SPFs, like all regression models, estimate the mean value of a dependent variable as a function of a set of independent variables. In the SPFs developed for the HSM, the dependent variable is the crash frequency for a roadway segment or intersection under base conditions, and the independent variables are the AADTs of the roadway segment or intersection legs (and, for roadway segments, the length of the roadway segment).

For rural conditions, different SPFs are provided for two- and four-lane highways and for three- and four-leg intersections. For urban and suburban arterials, prediction equations are provided for three-leg and four-leg intersections. Separate urban and suburban prediction equations are not provided based on the number of lanes on the major-road approach.

The accident modification factor for left-turn lanes is available from the *Highway Safety Manual*. For this evaluation, the intersections were assumed to have a stop sign on the minor approaches and that only one of the major-road approaches would be treated with a left-turn lane. The AMFs for both the rural two-lane and four-lane highway scenarios are 0.56 for three-leg intersections and 0.72 for four-leg intersections. The AMFs for the urban and suburban scenarios are 0.67 for three-leg intersections and 0.73 for four-leg intersections.

In 2008, a memo was released by the U.S. Department of Transportation regarding the treatment of the economic value of a statistical life in developmental analyses. The memo "raises to $5.8 million the value of a statistical life to be used by analysts in the Department of Transportation when assessing the benefit of preventing fatalities." The cost per crash requires knowing the distribution of crash severity, which was available in the *Highway Safety Manual*. Also needed is a conversion of the cost per person to a cost per crash. The number of individuals killed or injured in a crash is not readily available; however, values found in the literature or other projects were reasonable for use in this evaluation. The *Highway Safety Manual* also provides crash cost estimates by crash severity. These values were converted to 2009 dollars, and then the typical crash costs by number of legs and rural or urban area were determined. The HSM values represent costs per crash, while the value of a statistical life represents cost per person and had to be converted to cost per crash. The crash cost values in the *Highway Safety Manual* are assumed to already have accounted for the number of persons typically involved in a crash along with the distribution of injuries within a crash. The HSM values, however, do not account for the higher value being placed on a statistical life from the 2008 memo. A comparison is made between the two approaches as part of this project.

The typical crash cost per number of legs and rural or urban roadway based on the 2008 U.S. DOT memo, which states that the economic value of a statistical life is $5.8 million, is:

- $214,000 for three-leg rural roadways,
- $198,000 for four-leg rural roadways,
- $167,000 for three-leg urban and suburban roadways, and
- $180,000 for four-leg urban and suburban roadways.

The typical crash cost per number of legs and rural or urban roadway based on HSM data is:

- $129,000 for three-leg rural roadways,
- $135,000 for four-leg rural roadways,
- $113,000 for three-leg urban and suburban roadways, and
- $121,000 for four-leg urban and suburban roadways.

Typical construction costs for left-turn lanes were identified from several sources including previous literature and state department of transportation websites. A reasonable range for the cost of constructing a left-turn lane appears to be $100,000 to $375,000, with an average value of $250,000.

The above values were used in the benefit-cost evaluation to identify volume conditions when the installation of a left-turn lane at unsignalized intersections and major driveways would be cost-effective. Plots and tables were developed that indicate combinations of major-road traffic and left-turn lane volume where a left-turn lane would be recommended. Because the *Highway Safety Manual* prediction equations are not a function of operating speed or speed limit, the warrants are not separated by speed. The warrants are a function of the number of legs at the intersection along with whether it is in a rural or urban/suburban area. Warrants were developed using the following:

- A range of values for the economic value of a statistical life,
- Crash costs based on values in the *Highway Safety Manual*,
- A range of construction costs, and
- Benefit-cost ratio of 1.0 and 2.0.

The research team suggests a benefit-cost ratio of 1.0 along with the mid-range economic value of a statistical life and moderate construction cost to identify the warrants for a minimum left-turn treatment. For urban and suburban areas and multilane rural highways, that is a left-turn lane. For two-lane rural highways, that is a bypass lane. The benefit-cost ratio of 2.0 has been argued as being a more practical value to offset the potential variability in other assumptions. The warrants based on a benefit-cost ratio of 2.0 were selected for a left-turn lane on rural two-lane highways. These values were similar to the warrants that resulted when the lower crash costs based on older *Highway Safety Manual* values were used.

Economic Analysis for New Sites

There is an inherent difference between adding a left-turn lane at an existing site and constructing a new left-turn lane at a proposed development. While private property enjoys the right of access to the general system of public roadways, this is not an unlimited right. The right of access must be balanced with the needs of and potential harm to the general traveling public. In order to preserve mobility and provide safety for the traveling public, many transportation agencies have established regulations and programs to manage access to their roadway network. Agencies may require that steps be taken by a developer to mitigate projected traffic operations and/or safety impacts. An example of mitigation would be providing a left-turn lane to remove the traffic turning left into the site from the through arterial lanes. Many transportation agencies have the authority to require a developer to pay for this mitigation as long as there is a rational nexus between the projected impacts of the development and the needed improvements. In this manner, taxpayers do not have to pay for an improvement that may benefit predominantly one property owner. The findings from this study can be used to estimate the additional costs to a site if a new development causes left-turn volumes. The information available from this project reflects certain conditions, such as no cross minor-road volumes for a four-leg intersection. Simulation for a specific site would be needed to determine the expected increase in delay for the given intersection characteristics. The *Highway Safety Manual* can provide information regarding crashes.

CONCLUSIONS

Left-turn lanes can reduce the potential for collisions and improve capacity by removing stopped vehicles from the main travel lane. The conclusions and recommendations developed based on this research are:

- Left-turn lane warrants were developed using an economic analysis procedure for:
    - Rural two-lane highways (see Table 81and Figure 40),
    - Rural four-lane highways (see Table 82 and Figure 41), and
    - Urban and suburban roadways (see Table 80 and Figure 39).
- The methodology presented in this report could also be used if a transportation agency has available local values for delay reductions due to the installation of a left-turn lane, crash frequency or crash predictions, crash reduction factors, crash costs, and/or construction costs.
- Methodology is also presented in this report that can be used to estimate the impacts due to a new development. Impacts discussed include increased delay due to the new development along with predicted safety consequences. The example provided is for conditions assumed to generate the delay value used in this project. Simulation for a specific site would be needed to determine the expected increase in delay for the given intersection characteristics. The *Highway Safety Manual* provides safety performance functions for predicting the number of crashes.
- A legal review conducted as part of this research addressed the following question: When a government seeks to fulfill a broad public objective such as safety and, in this project,

left-turn accommodation, who should bear the costs—the developer who would be adding traffic to the roadway network or the general public? Unless the facts of the case are clear cut, the outcome is unpredictable even within specific jurisdictions. Results will be more predictable if agencies have the documented authority to manage access.

APPENDIX A

REVISED TEXT ON LEFT-TURN LANE WARRANTS FOR THE AASHTO *GREEN BOOK*

This appendix presents revised text on left-turn lane warrants for consideration by AASHTO for inclusion in the AASHTO *A Policy on Geometric Design of Highways and Streets*. Material suggested for removal is shown with a strikethrough line. Double underlines indicate material to be added. Figures or tables beginning with the number 9 are from the AASHTO *Green Book*, while those beginning with the letter A reflect recommended additions to the text.

AASHTO *GREEN BOOK* (2004), RURAL AND URBAN ARTERIALS (URBAN), PAGES 488 TO 490

Operational and Control Measures for Left-Turn Maneuvers

Vehicles turning left into cross streets or all mid-block locations may cause substantial delays to through traffic and may contribute to crashes, thus diminishing arterial effectiveness. There is a popular belief that the effects of such left-turn movements can be eliminated simply by placing "No Left Turn" signs. In fact, motorists that desire to turn left do not just disappear, but reach their destinations by alternative routes. Thus, prohibition of left turns at some locations may create or increase operational or safety problems at other locations.

Effective control of turning movements lies in discovering or anticipating the extent of the problem and in providing for the movements through a combination of measures including selective prohibition or turns, geometric design, and traffic control. It is difficult to discuss these factors independently, and no firm rules are applicable to all situations. Several principles and methods that, if properly considered and applied, will lead to appropriate designs are outlined as follows:

1 The capability for motorists to reach their desired destinations must be provided. Left turns should not be prohibited unless alternative routings are available.

2 As a general rule, the fewer the number of left turns at any location, the less the interference with other traffic. Thus, for a given total number of left turns within a given length of highway, it may be better to encourage a few left turns at each of several locations than to concentrate the turns at a single location.

3 Separate signal phases for left-turn movements reduce the amount of green time available for other movements at the intersection. Multiphase signals are therefore advantageous only if traffic operation and safety are improved sufficiently to offset the loss in green time. This determination should be made on a case-by-case basis.

4 Where selective prohibition of left turns is necessary, there are operational advantages in concentrating left turns at intersections where the volume of cross traffic is low so that a

large fraction of the signal time is available for the green phase on the arterial. Where two arterial streets intersect, there may be advantage in requiring left-turning vehicles to bypass the main intersection. For instance, one manner in which the left-turn maneuver from one arterial to another can be accomplished is to require the motorist to turn left from the first arterial a block in advance of the main intersection, then proceed one block, turn right, proceed another block, and turn left. Where such techniques are used, clear guide signing is essential.

5 It is sometimes advantageous to route left-turning traffic around a block, through a series of right turns after passing through the main intersection, rather than permitting a direct left-turn maneuver. However, this approach has disadvantages as well. Traffic volumes are increased because the left-turning vehicle now must pass through the intersection twice. In addition, the distance of travel by the vehicle that desires to turn left is increased, and the increased right-turn volumes may have an impact on the operation of three other intersections. This approach to left-turn maneuvers should generally be limited to locations where the left-turn volumes are small and the provision of a separate left-turn lane is not practical.

6 The desirability of exclusive left-turn lanes cannot be overemphasized. Such lanes may consist of separate left-turn lanes in the median or continuous center lanes used exclusively for left turns from both directions. Economic analysis was used to identify situations for an unsignalized intersection in which left-turn lanes are and are not justified based on delay savings and crash reductions. The warrants are presented in the Intersection Chapter. ~~Multiphase signal control is very inefficient if turning traffic and through traffic both use the same lane. Where turning traffic is light, a left-turn lane may eliminate the need for the left-turn signal phasing because the storage of left-turning vehicles will not affect through traffic. Traffic safety is greatly enhanced if turning vehicles can be stored separately from lanes used by through vehicles.~~

7 Multiphase signal control is very inefficient if turning traffic and through traffic both use the same lane. Where turning traffic is light, a left-turn lane may eliminate the need for the left-turn signal phasing because the storage of left-turning vehicles will not affect through traffic. Traffic safety is greatly enhanced if turning vehicles can be stored separately from lanes used by through vehicles.

~~7~~8 With a separate left-turn phase, dual left-turn lanes can accommodate up to about 180 percent of the volume that can be served by a single left-turn lane with the same available green time, depending on the width of the cross street and the radius of turn. Desirably, the turning radius for a dual left-turn lane is 27 m (90 ft). Thus, where sufficient right-of-way space for a long-radius turn, and a wide cross street are available, the installation of dual left-turn lanes may be a practical design to serve a heavy left-turn movement. Exhibit 7-15 shows an example of dual left-turn lanes at an intersection on an urban arterial. Further guidance concerning the design of dual left-turn lanes is presented in Chapter 9 and in the HCM (2).

89 Grade separations or other special treatments for left-turn movements are sometimes appropriate, as discussed in Chapter 10.

In summary, left-turn demands should be accommodated as near as practical to the point of which the motorist desires to turn left. Shifting the left-turn maneuvers away from this point of desire may lead to secondary problems. Nevertheless, if the point at which motorists desire to turn left is highly objectionable from the standpoint of design, traffic control, or safety, regulatory measures may be employed to move those left turns to a location that is more suitable. Only in exceptional cases should such maneuvers be shifted more than two blocks from the point of desire. Where left turns are permitted from an arterial street, the intersection design should incorporate left-turn storage lanes unless it is impractical to provide them.

AASHTO *GREEN BOOK* (2004), INTERSECTION CHAPTER, PAGES 488 TO 490

GENERAL INTERSECTION TYPES
General Design Considerations

General types of intersections and terminology are indicated in Exhibits 9-73 and 9-74. The geometric forms are the three-leg, four-leg, and multileg intersections. Further classification includes such variations as unchannelized, flared, and channelized intersections. Details and specific adaptations of each general type are demonstrated in the section of this chapter on "Types and Examples of Intersections."

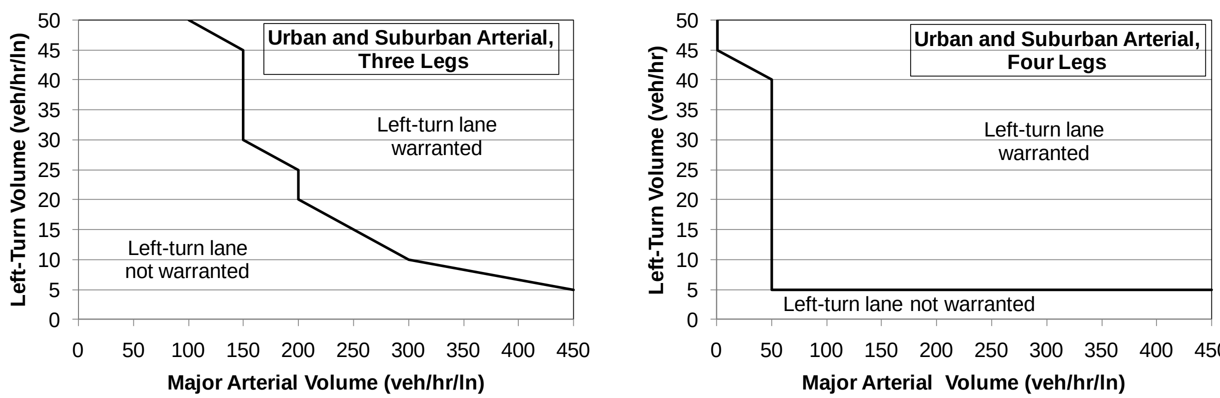
Many factors enter into the choice of type of intersection and the extent of design of a given type, but the principal controls are the design-hour traffic volume, the character or composition of traffic, and the design speed. The character of traffic and design speed affect many details of design, but in choosing the type of intersection they are not as significant as the traffic volume. Of particular significance are the actual and relative volumes of traffic involved in various turning and through movements.

When designing an intersection, left-turning traffic should be removed from the through lanes, whenever practical. Therefore, provisions for left turns (i.e., left-turn lanes) have widespread application. Ideally, left-turn lanes should be provided at driveways and street intersections along major arterial and collector roads wherever left turns are permitted. In some cases or at certain locations, providing for indirect left turns (jughandles, U-turn lanes, and diagonal roadways) may be appropriate to improve safety and preserve capacity. The provision of left-turn lanes has been found to reduce crash rates anywhere from 20 to 65 7 to 48 percent (*Highway Safety Manual*) (1). Left-turn facilities should be established on roadways where traffic volumes are high enough or safety considerations are sufficient to warrant them. They are often needed to ensure adequate service levels for the intersections and the various turning movements.

Guidelines for when left-turn lanes should be provided are set forth in Table A-1 for urban and suburban roadways and Tables A-2 and A-3 for rural highways (and Figure A-1, A-2, and A-3 for urban and suburban roadways, rural two-lane highways, and rural four-lane highways, respectively). Ssseveral documents for both signalized and unsignalized intersections provide guidance on left-turn lanes (11, 12, 13, *NCHRP 3-91 Design Guide on Left-Turn Accomodations* *at Unsignalized Intersections,* FHWA *Signalized Intersections: Informational Guide,* FHWA-HRT-04-091). ~~These guidelines key the need for left-turn lanes to (a) the number of arterial lanes, (b) design, and operating speeds, (c) left-turn volumes, and (d) opposing traffic volumes.~~

Table A-1. Suggested left-turn lane warrants based on results from benefit-cost evaluations for urban and suburban arterials.

Left-Turn Lane Peak-Hour Volume (veh/hr)	**Three-Leg Intersection, Major Urban and Suburban Arterial Volume (veh/hr/ln) That Warrants a Left-Turn Lane**	**Four-Leg Intersection, Major Urban and Suburban Arterial Volume (veh/hr/ln) That Warrants a Left-Turn Lane**
5	450	50
10	300	50
15	250	50
20	200	50
25	200	50
30	150	50
35	150	50
40	150	50
45	150	< 50
50 or More	100	< 50

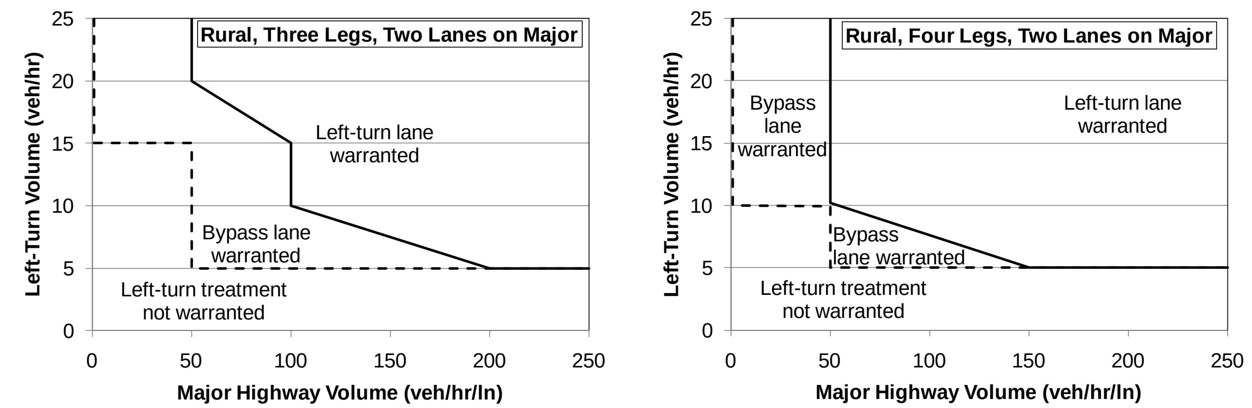


(a) Three Legs **(b) Four Legs**

Figure A-1. Suggested left-turn lane warrants based on results from benefit-cost evaluations for intersections on urban and suburban arterials.

Table A-2. Suggested left-turn treatment warrants based on results from benefit-cost evaluations for rural two-lane highways.

Left-Turn Lane Peak-Hour Volume (veh/hr)	**Three-Leg Intersection, Major Two-Lane Highway Peak-Hour Volume (veh/hr/ln) That Warrants a Bypass Lane**	**Three-Leg Intersection, Major Two-Lane Highway Peak-Hour Volume (veh/hr/ln) That Warrants a Left-Turn Lane**	**Four-Leg Intersection, Major Two-Lane Highway Peak-Hour Volume (veh/hr/ln) That Warrants a Bypass Lane**	**Four-Leg Intersection, Major Two-Lane Highway Peak-Hour Volume (veh/hr/ln) That Warrants a Left-Turn Lane**
5	50	200	50	150
10	50	100	≤ 50	50
15	≤ 50	100	≤ 50	50
20	≤ 50	50	≤ 50	≤ 50
25	≤ 50	50	≤ 50	≤ 50
30	≤ 50	50	≤ 50	≤ 50
35	≤ 50	50	≤ 50	≤ 50
40	≤ 50	50	≤ 50	≤ 50
45	≤ 50	50	≤ 50	≤ 50
50 or More	≤ 50	50	≤ 50	≤ 50

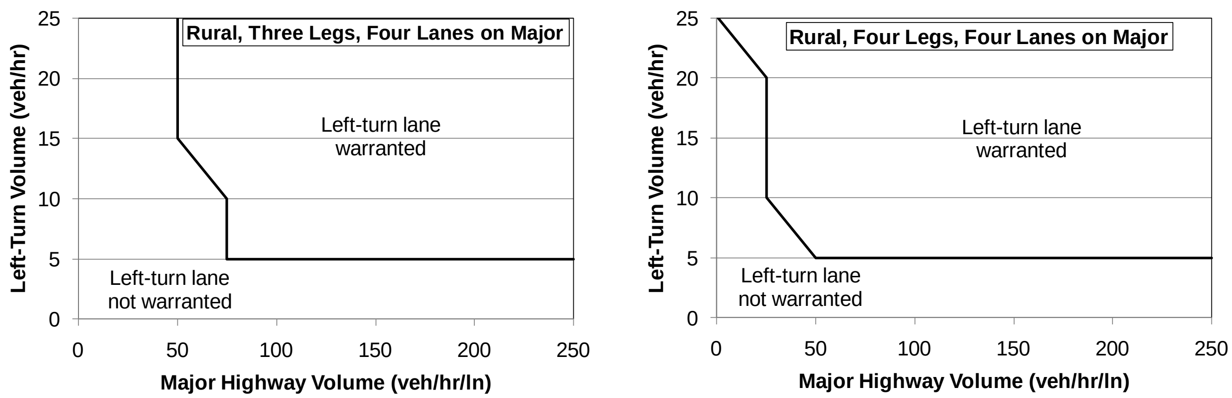


(a) Three Legs **(b) Four Legs**

Figure A-2. Suggested left-turn treatment warrants based on results from benefit-cost evaluations for intersections on rural two-lane highways.

Table A-3. Suggested left-turn lane warrants based on results from benefit-cost evaluations for rural four-lane highways.

Left-Turn Lane Peak-Hour Volume (veh/hr)	Three-Leg Intersection, Major Four-Lane Highway Peak-Hour Volume (veh/hr/ln) That Warrants a Left-Turn Lane	Four-Leg Intersection, Major Four-Lane Highway Peak-Hour Volume (veh/hr/ln) That Warrants a Left-Turn Lane
5	75	50
10	75	25
15	50	25
20	50	25
25	50	< 25
30	50	< 25
35	50	< 25
40	50	< 25
45	50	< 25
50 or More	50	< 25



(a) Three Legs **(b) Four Legs**

Figure A-3. Suggested left-turn lane warrants based on results from benefit-cost evaluations for intersections on rural four-lane highways.

The HCM (6) indicates that exclusive left-turn lanes at signalized intersections should be installed as follows:

- Where fully protected, left-turn phasing is to be provided;
- Where space permits, left-turn lanes should be considered when left-turn volumes exceed 100 veh/hr (left-turn lanes may be provided for lower volumes as well as on the basis of the judged need and state of local practice, or both); and
- Where left-turn volumes exceed 300 veh/hr, a double left-turn lane should be considered.

Exhibit 9-75 is a guide to traffic volumes where left-turn facilities should be considered on two-lane highways. For the volumes shown, left turns and right turns from the minor street can be equal to, but not greater than, the left turns from the major street.

Additional information on left-turn lanes, including their suggested lengths, can be found in published sources (2, 11, 13, *NCHRP Project 3-91 Design Guide on Left-Turn Accommodations at Unsignalized Intersections*). In the case of double left-turn lanes, a capacity analysis of the intersection should be performed to determine what traffic controls are needed in order for it to function properly.

Local conditions and the cost of right-of-way often influence the type of intersection selected as well as many of the design details. Limited sight distance, for example, may make it desirable to control traffic by yield signs, stop signs, or traffic signals when the traffic densities are less than those ordinarily considered appropriate for such control. The alignment and grade of the intersecting roads and the angle of intersection may make it advisable to channelize or use auxiliary pavement areas, regardless of the traffic densities. In general, traffic service, highway design designation, physical conditions, and cost of right-of way are considered jointly in choosing the type of intersection.

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

Exhibit 9-75. Guide for Left-Turn Lanes on Two-Lane Highways (6)

For the general benefit of through-traffic movements, the number of crossroads, intersecting roads, or intersecting streets should be minimized. Where intersections are closely spaced on a two-way facility, it is seldom practical to provide signals for completely coordinated traffic movements at reasonable speeds in opposing directions on that facility. At the same time the resultant road or street patterns should permit travel on roadways other than the predominant highway without too much inconvenience. Traffic analysis is needed to determine whether the road or street pattern, left open across the predominate highway, is adequate to serve normal traffic plus the traffic diverted from any terminated road or street.

The functional classification of the road, the patterns of traffic movement at the intersections, and the volume of traffic on each approach, including pedestrians, during one or more peak periods of the day are indicative of the type of traffic control devices necessary, the roadway widths needed (including auxiliary lanes), and, where applicable, the degree of channelization needed to expedite the movement of all traffic. The differing arrangement of islands and the shape and length of auxiliary lanes depend on whether signal control is provided.

The composition and character of traffic are a design control. Movements involving large trucks need larger intersection areas and flatter approach grades than those needed at intersections where traffic consists predominantly of passenger cars. Bus stops located near an intersection may further modify the arrangement. Approach speeds of traffic also have a bearing on the geometric design as well as on control devices and markings.

The number and locations of the approach roadways and their angles of intersection are major controls for the intersection geometric pattern, the location of islands, and the types of control devices. Intersections preferably should be limited to no more than four approach legs. Two or more crossroads intersecting an arterial highway in close proximity should be combined into a single crossing.

The distance between intersections influences the degree of channelization at any one particular intersection. For example, where intersections are closely spaced, turn restrictions may be imposed at some intersections and pedestrian crossings may be prohibited at others. This makes some channelizing islands and auxiliary pavement areas unnecessary, or it may be appropriate to introduce continuous auxiliary lanes between two or more intersecting roads or streets to handle a buildup and weaving of traffic. Where crossroads are widely spaced, each intersection should accommodate all crossing, turning, and pedestrian movements.

APPENDIX B

REVISED TEXT ON LEFT-TURN LANE WARRANTS FOR THE TRB *ACCESS MANAGEMENT MANUAL*

Following are sections from the TRB *Access Management Manual*. Material suggested for removal is shown with a strikethrough line. Double underlines indicate material to be added. Figures or tables beginning with the number 10 are from the TRB *Access Management Manual*, while those beginning with the letter B reflect recommended additions to the text.

ACCESS MANAGEMENT MANUAL (2003), CHAPTER 10, ACCESS DESIGN, AUXILIARY LANES (PAGES 171 TO 177)

AUXILIARY LANES

An auxiliary lane (left-turn and right-turn bays) is the most effective means of limiting the speed differential between a turning vehicle and following through traffic to a safe level. This section addresses considerations in the decision of when to provide a left-turn lane; in the design of auxiliary lanes, including maneuver distance, queue storage, determination of turn bay length, and providing for dual left turns; and in the design of right-turn storage. Guidance for when to provide left-lane lanes was developed in NCHRP Project 3-91, Left-Turn Accommodations at Unsignalized Intersections.

The minimum physical length of a right-turn or left-turn bay, including the taper, consists of the maneuver distance plus the queue storage (distance *d2*, *d3*) (Figure 10-5). (The distance to maneuver laterally and decelerate to a stop, *d*2, is the same for left-turn bays as for right-turn bays, because the initial speed and the speed at which the turning vehicle clears the through-traffic lane are the same.) Additional details on the design of left-turn and right-turn bays are provided in *Transportation and Land Development* (*3*) and the participant's notebook from National Highway Institute Course 133078 (*1*).

Warrants for Left-Turn Lanes

Economic analysis was used to identify situations in which left-turn lanes are and are not justified at an unsignalized intersection based on delay savings and crash reductions (reference to NCHRP Project 03-91). Guidelines for when left-turn lanes should be provided are set forth in Table B-1 and Figure B-1 for urban and suburban roadways, Table B-2 and Figure B-2 for rural two-lane highways, and Table B-3 and Figure B-3 for rural four-lane highways.

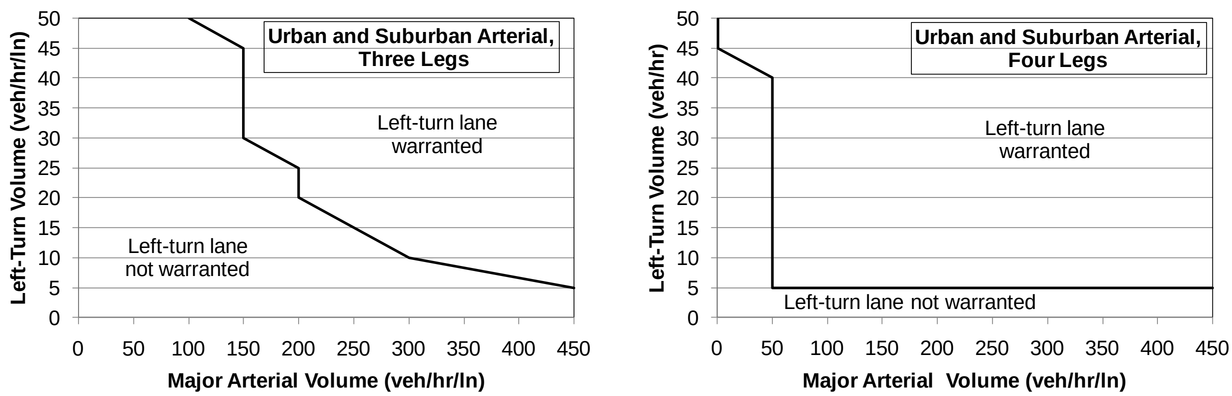
The *Highway Capacity Manual* (reference) indicates that exclusive left-turn lanes at signalized intersections should be installed as follows:

- Where fully protected, left-turn phasing is to be provided;
- Where space permits, left-turn lanes should be considered when left-turn volumes exceed 100 veh/hr (left-turn lanes may be provided for lower volumes as well as on the basis of the judged need and state of local practice, or both); and

- Where left-turn volumes exceed 300 veh/hr, a double left-turn lane should be considered.

Table B-1. Suggested left-turn lane warrants based on results from benefit-cost evaluations for urban and suburban arterials.

Left-Turn Lane Peak-Hour Volume (veh/hr)	**Three-Leg Intersection, Major Urban and Suburban Arterial Volume (veh/hr/ln) That Warrants a Left-Turn Lane**	**Four-Leg Intersection, Major Urban and Suburban Arterial Volume (veh/hr/ln) That Warrants a Left-Turn Lane**
5	450	50
10	300	50
15	250	50
20	200	50
25	200	50
30	150	50
35	150	50
40	150	50
45	150	≤ 50
50 or More	100	≤ 50

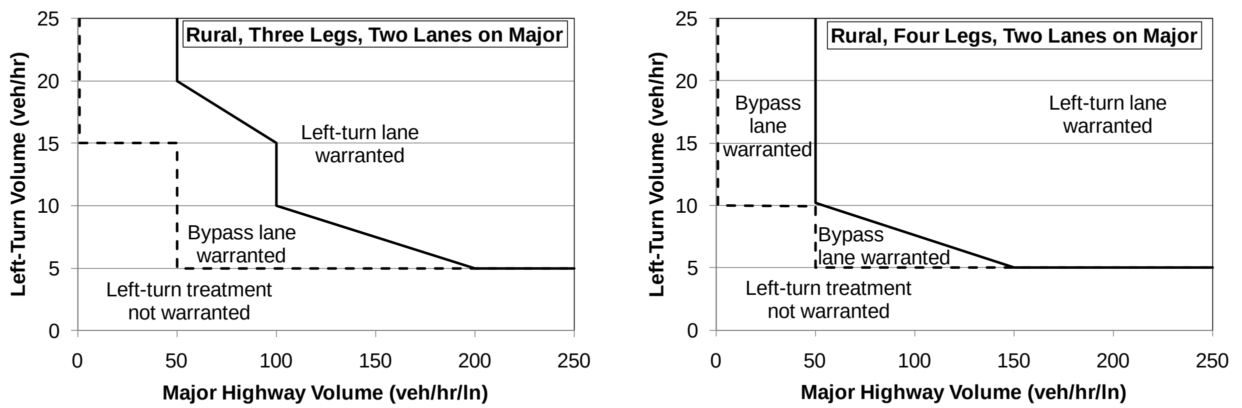


(a) Three Legs **(b) Four Legs**

Figure B-1. Suggested left-turn lane warrants based on results from benefit-cost evaluations for intersections on urban and suburban arterials.

Table B-2. Suggested left-turn treatment warrants based on results from benefit-cost evaluations for rural two-lane highways.

Left-Turn Lane Peak-Hour Volume (veh/hr)	**Three-Leg Intersection, Major Two-Lane Highway Peak-Hour Volume (veh/hr/ln) That Warrants a Bypass Lane**	**Three-Leg Intersection, Major Two-Lane Highway Peak-Hour Volume (veh/hr/ln) That Warrants a Left-Turn Lane**	**Four-Leg Intersection, Major Two-Lane Highway Peak-Hour Volume (veh/hr/ln) That Warrants a Bypass Lane**	**Four-Leg Intersection, Major Two-Lane Highway Peak-Hour Volume (veh/hr/ln) That Warrants a Left-Turn Lane**
5	50	200	50	150
10	50	100	≤ 50	50
15	≤ 50	100	≤ 50	50
20	≤ 50	50	≤ 50	≤ 50
25	≤ 50	50	≤ 50	≤ 50
30	≤ 50	50	≤ 50	≤ 50
35	≤ 50	50	≤ 50	≤ 50
40	≤ 50	50	≤ 50	≤ 50
45	≤ 50	50	≤ 50	≤ 50
50 or More	≤ 50	50	≤ 50	≤ 50

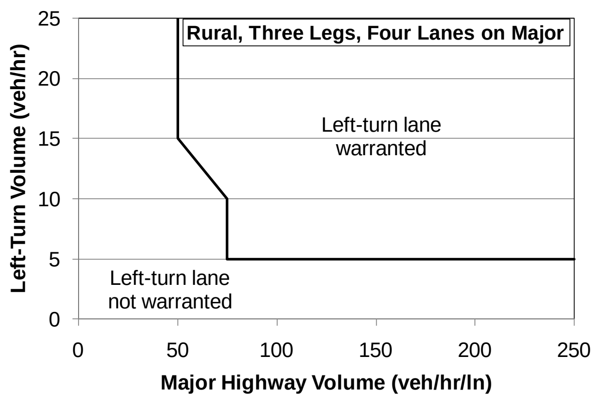


(a) Three Legs **(b) Four Legs**

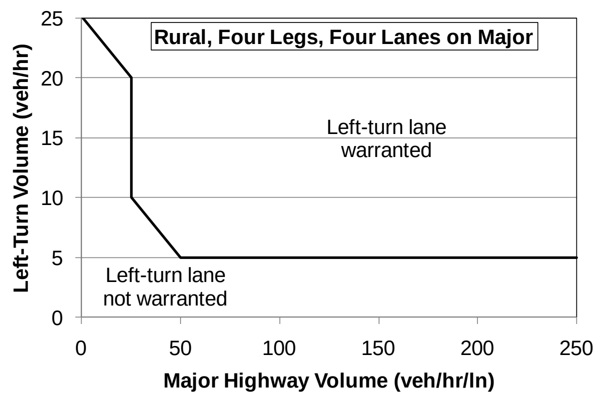
Figure B-2. Suggested left-turn treatment warrants based on results from benefit-cost evaluations for intersections on rural two-lane highways.

Table B-3. Suggested left-turn lane warrants based on results from benefit-cost evaluations for rural four-lane highways.

Left-Turn Lane Peak-Hour Volume (veh/hr)	Three-Leg Intersection, Major Four-Lane Highway Peak-Hour Volume (veh/hr/ln) That Warrants a Left-Turn Lane	Four-Leg Intersection, Major Four-Lane Highway Peak-Hour Volume (veh/hr/ln) That Warrants a Left-Turn Lane
5	75	50
10	75	25
15	50	25
20	50	25
25	50	≤ 25
30	50	≤ 25
35	50	≤ 25
40	50	≤ 25
45	50	≤ 25
50 or More	50	≤ 25



(a) Three Legs



(b) Four Legs

Figure B-3. Suggested left-turn lane warrants based on results from benefit-cost evaluations for intersections on rural four-lane highways.

Maneuver Distance

Table 10-2 presents the estimated distances that would be needed by drivers to maneuver from the through lane into a turn bay and brake to a stop. These distances provide for a 10-mph (15-km/h) difference in speed for the turning vehicle when it clears the through lane. Shorter length will result in a speed differential greater than 10 mph (15 km/h).

Table 10-2 Desirable Maneuver Distances (*d2* in Figure 10-5)

Speed		Distance[a]	
mph	**km/h**	**ft**	**m**
20	30	70	20
30	50	160	45
40	65	275	85
50	80	425	130
60	95	605	185
70	110	820	245

Note:
1. assumes a turning vehicle has "cleared the through lane" when it has moved laterally about 9 ft (3 m) so that a following through vehicle can pass without encroaching upon the adjacent traffic lane.
2. The speed differential between the turning vehicle and following through vehicles is 10 mph (15 km/h) when the turning vehicle "clears the through traffic lane."
3. 5.8 ft/s^2 deceleration while moving from the through lane into the turn lane; 6.5 ft/s^2 average deceleration after completing lateral shift into the turn lane.
[a] Rounded to 5 ft (5 m)

Queue Storage

A turn bay needs to be of adequate length to store vehicles waiting to complete the turn (distance *d*, in Figure 10-5). The number of left-turning vehicles arriving at an intersection will vary from one short time interval to another. Figure 10-6 illustrates this characteristic for a series of 2-min intervals. For this example, the average number of left turns per cycle is two vehicles, where the number of left turns on any given cycle ranges from none to four. It is important for turn bays on roadways of a high functional classification to be of sufficient length to store all arriving vehicles most of the time. On roadways of lesser importance, a lower likelihood of storing all arriving vehicles may be acceptable. The left-turn storage length at a signalized intersection will be affected by:

- The left-turn flow rate for the design hour;
- Signal timing, including the cycle length (number of cycles per hour), duration of a separate left-turn phase (left-turn arrow, if left turns are permitted on "green" or if left turns are restricted to "left on green arrow only"), and the number of left-turn lanes;
- The probability the storage length will be sufficient to store all left-turning vehicles on an acceptable percentage of the cycles, such as at least 99.5% or 95% of the time for a ~~at the intersection on a~~ major arterial or a collector; and
- The percentage of large vehicles.

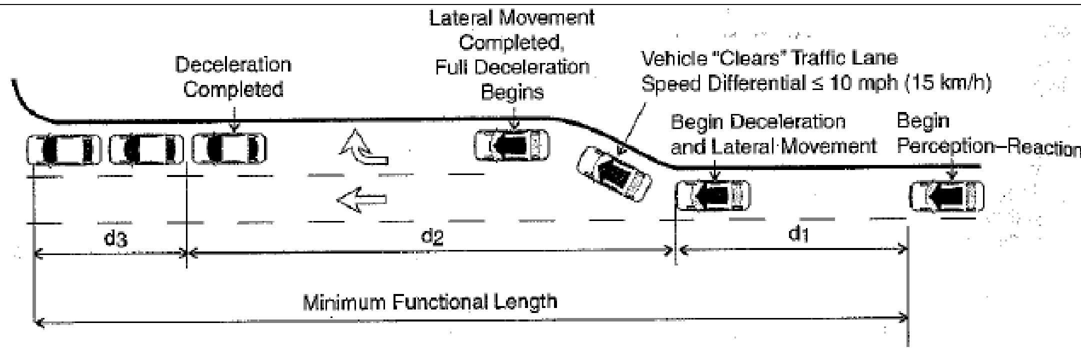


FIGURE 10-5 Upstream functional intersection area (*1*, *3*). (d_1 = distance traveled during perception–reaction time; d_2 = distance traveled while driver maneuvers laterally and decelerates to a stop; d_3 = storage length.)

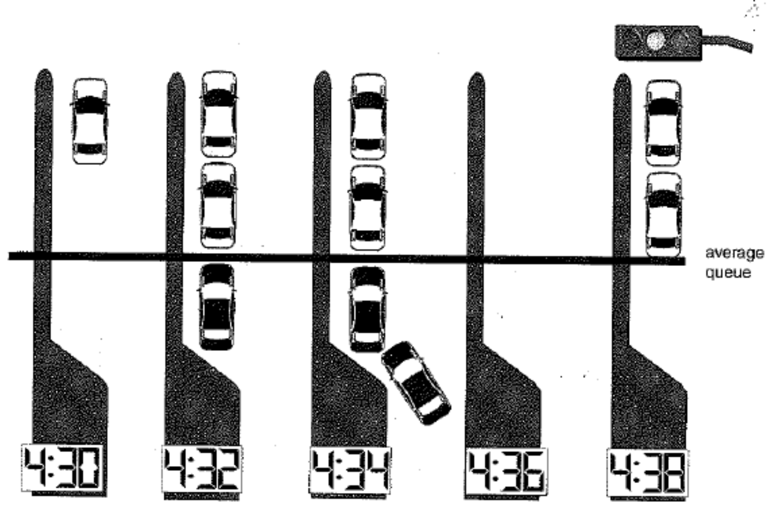
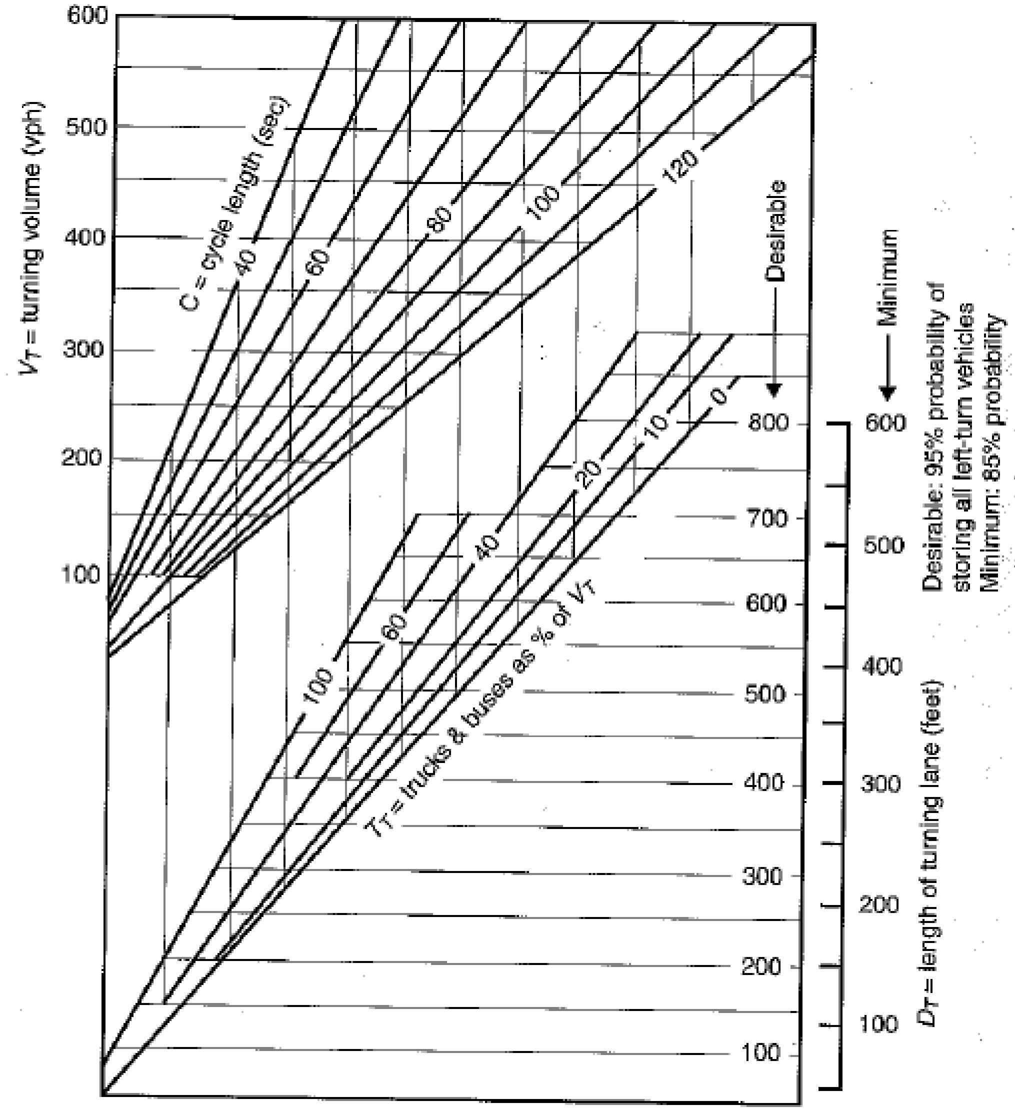


FIGURE 10-6 Illustration of the need to store more than the average number of left-turning vehicles.

Figure 10-7 presents an easy to use nomograph for estimating minimum left-turn storage. The same procedure can be used for left turns at unsignalized intersections by assuming a time interval (in lieu of cycle length). AASHTO (*6*, 1990, p. 829) suggests using a 2-min interval, which is the practice of the Florida Department of Transportation.



Example

Conditions	Solution
• Left-turn volume, V_T = 240 vph	• Desirable storage = 400 ft
• 120-second cycle length	• Minimum storage = 300 ft
• No trucks or buses	

FIGURE 10-7 Single-lane left-turn queue storage at signalized intersections (*10*).

The following equation is another easy method for estimating left-turn queue storage length:

$$ - \qquad (1) $$

Where

L = design length for left-turn storage (ft);
V = estimated left-turn volume [vehicles per hour (veh/hr)];
N = number of cycles per hour (for the *Green Book* unsignalized procedure this would be 30) (*V/N* is the average number of turning vehicles per cycle);
k = factor that is the length of the longest queue (design queue length) divided by average queue length (a value of 2.0 is commonly used for major arterials and a value of 1.5 to 1.8 might be considered for an approach on a minor street or on a collector where capacity will not be critical) (for the *Green Book* procedure this would be 1); and
s = average length per vehicle, including the space between vehicles, generally assumed to be 25 ft (7.6 m) (if percentage of trucks and buses is known, minimum queue storage can be adjusted by using Table 10-3).

Table 10-3 Queue Storage Length Adjustments for Trucks

Percent Trucks	Adjustments	
	ft	m
≤ 5	25	7.6
10	30	9.0
15	35	10.7

Bonneson et al. suggest guidelines for determining if the bay storage length for an unstopped approach is adequate in *NCHRP Report 457*. They use 4.1 sec for critical gap and 2.2 sec for follow-up time. Using their approach along with the critical gaps identified as part of NCHRP Project 3-91 results in the storage length recommendations listed in Table B-4. The approach assumes a 0.005 probability of overflow.

Table 10-4 presents a comparison of the queue lengths obtained from Leisch's nomograph (cf. Figure 10-7), ~~and~~ Equation 1, and the updated Bonneson approach. The nomographs developed by Harmelink in 1967 (*7*), or variations thereof, are often used to estimate queue storage length at unsignalized access connections. These curves and their variations (*6, 11*) have been widely used. The updated Bonneson approach considers the critical gaps observed in more recent research and allows sensitivity to the opposing volume quantity. ~~However, recent information indicates that the time intervals for perception–reaction time plus the maneuver time to complete the left-turn maneuver are consistently larger than that used in developing the curves in 1967. Hence, left-turn bays would be warranted at lower volumes and storage lengths would need to be larger. A discussion of this problem is included in the participant's notes for National Highway Institute Course 133078 (*1*) and *Transportation and Land Development* (*3*).~~

Table B-4. Recommended Storage Lengths for Arterials from *Access Management Manual* Equation and *NCHRP Report 457* Equations with Revised Critical Gap.

Left-Turn Volume (veh/hr)	Storage Length, Rounded Up to Nearest 25-ft Increment (ft)						
	Storage Lengths from Other Manuals for Comparison		Storage Lengths Calculated from Equations [b] Documented in *NCHRP Report 457* Using Revised Critical Gaps and 0.005 Probability of Overflow				
	Green Book Procedure (k=1)[a]	Equation (k=2)[a]	Opposing Volume (veh/hr)				
			200	400	600	800	1000
Critical Gap = 5.0 sec, Follow-Up Gap = 2.2 sec (Represents the 50th Percentile Critical Gap Found in Field Studies)							
40	75	75	50	50	50	50	50
60	50	100	50	50	50	50	50
80	75	150	50	50	50	50	50
100	100	175	50	50	50	50	75
120	100	200	50	50	50	75	75
140	125	250	50	50	50	75	75
160	150	275	50	50	75	75	100
180	150	300	50	50	75	75	100
200	175	350	50	75	75	100	125
220	200	375	50	75	75	100	125
240	200	400	75	75	100	125	150
260	225	450	75	75	100	125	175
280	250	475	75	75	100	125	175
300	250	500	75	100	125	150	200
Critical Gap = 6.25 sec, Follow-Up Gap = 2.2 sec (Represents the 85th Percentile Critical Gap Found in Field Studies, 85th Percentile Is Preferred for Design)							
40	75	75	50	50	50	50	50
60	50	100	50	50	50	50	50
80	75	150	50	50	50	50	75
100	100	175	50	50	50	75	75
120	100	200	50	50	75	75	100
140	125	250	50	50	75	100	125
160	150	275	50	75	75	100	150
180	150	300	50	75	75	125	150
200	175	350	50	75	100	125	200
220	200	375	75	75	100	150	225
240	200	400	75	75	125	150	275
260	225	450	75	100	125	175	325
280	250	475	75	100	125	200	400
300	250	500	75	100	150	225	525

[a, b] See Table B-5 for equations.

This table assumes 25 ft per vehicle spacing. Table 10-3 provides other suggested spacing lengths based on percent trucks.

Table B-5. Equations Used to Determine Storage Length.

Equation in TRB *Access Management Manual*

(1)

Where

L = design length for left-turn storage (ft);
V = estimated left-turn volume, vehicles per hour (veh/hr);
N_c = number of cycles per hour (for the *Green Book* unsignalized procedure this would be 30) (V/N is the average number of turning vehicles per cycle);
k = factor that is the length of the longest queue (design queue length) divided by average queue length (a value of 2.0 is commonly used for major arterials, and a value of 1.5 to 1.8 might be considered for an approach on a minor street or on a collector where capacity will not be critical) (for the *Green Book* procedure this would be 1); and
s = average length per vehicle, including the space between vehicles, generally assumed to be 25 ft (adjustments are available in several documents for trucks and buses such as the TRB *Access Management Manual*—see Table 10-3).

Equations Used in *NCHRP Report 457*

Equations also used to generate values in Table B-4

Where

$P(n>N)$ = probability of bay overflow;
v = left-turn vehicle volume (veh/hr);
N = number of vehicle storage positions;
c = movement capacity (veh/hr);
V_o = major-road volume conflicting with the minor movement, assumed to be equal to one-half of the two-way major-road volume (veh/hr);
SL = storage length (ft);
t_c = critical gap (sec);
t_f = follow-up gap (sec); and
VL = average length per vehicle, including the space between vehicles, generally assumed to be 25 ft (adjustments are available in several documents for trucks and buses such as the TRB *Access Management Manual*—see Table 10-3).

Table 10-4 Comparison of Queue Storage

Method	Volume (veh/hr)	Cycle Length (s)	Storage (ft)	
			Desirable	Minimum
Nomograph	240 240	60 120	200 400	150 300
L=(V/N)(2.0)(25)	240 240	60 120	200 400	
L=(V/N)(1.5)(25)	240 240	60 120		150 300

Method	Volume (veh/hr)	Cycle Length (s)	Opposing Volume (veh/hr)	Storage (ft)	
				Desirable	Minimum
Nomograph	240 240	60 120		200 400	150 300
L=(V/N)(2.0)(25)	240 240	60 120		200 400	
L=(V/N)(1.5)(25)	240 240	60 120			150 300
Updated Bonneson Approach with Critical Gap = 5.0 sec	240	NA	200 400 600 800 1000	75 75 100 125 150	
Updated Bonneson Approach with Critical Gap = 6.25 sec	240	NA	200 400 600 800 1000	75 75 125 150 275	

The nomograph for four-lane roadways is presented in Figure 10-8. This nomograph is used by reading horizontally for the opposing volume *Vo* and vertically for the turning volume *VL*. The minimum storage length *S* is where the vertical and horizontal lines intersect. For example, for *Vo* = 750 and *VL* = 50, the minimum storage length is 75 ft. A vehicle making a left turn from a two-lane roadway blocks all following advancing vehicles. Therefore, a set of several nomographs is needed for two-lane roadways for different combinations of speed, opposing volume, advancing volume, and percent left turns. Table 10-5 presents in a simplified form minimum left-turn queue storage on two-lane roadways.

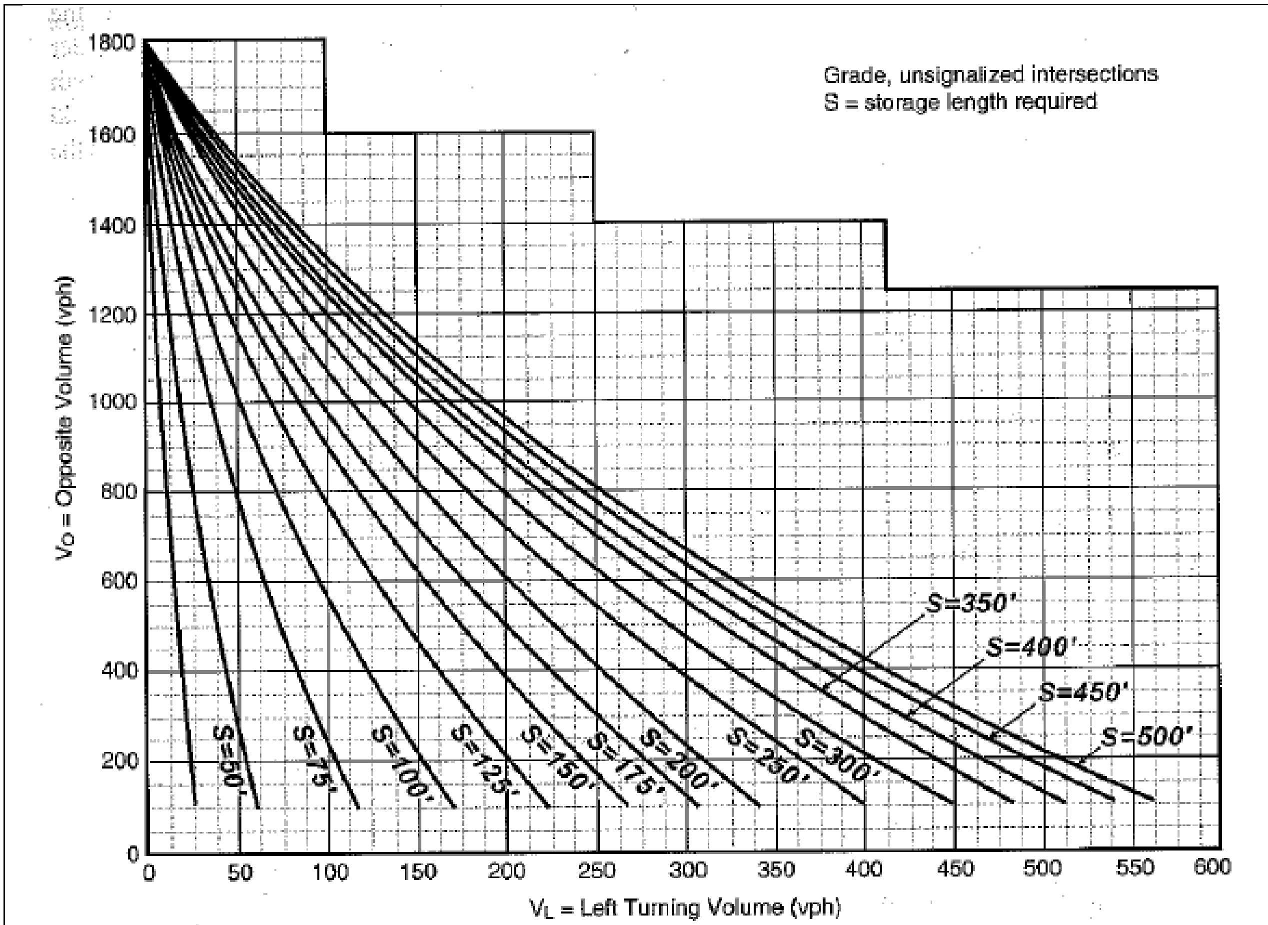


FIGURE 10-8 Minimum left-turn storage at unsignalized intersections on four-lane roadways (7).

~~When applying the storage length from Figure 10-8 and Table 10-5, as well as adaptations of the Harmelink nomographs, keep in mind the following. Recent research (*12*) indicates that the perception reaction time needed by most drivers is considerably longer than that used by Harmelink. The time to complete a left turn is also found to be longer (*6*).~~

Current practice indicates that a minimum storage length is at least 100 ft (30 m) in suburban/urban areas and 50 ft (15 m) in rural areas (*1*, *3*). These minimum lengths would apply when methods such as ~~the nomograph or~~ Equation 1 result in shorter lengths or when there are no traffic estimates upon which to determine queue length. Where dual left-turn lanes are provided, the storage length may be estimated by dividing the single-lane queue storage length by 1.8 and rounding to the next highest 25-ft interval.

~~Table 10-5 Minimum Left-Turn Queue Storage Lengths (ft) on Two-Lane Roadways~~

~~Opposing Volume (veh/hr)~~	~~Advancing Volume (veh/hr)~~			
	~~200~~	~~400~~	~~600~~	~~800~~
~~Speed = 40 mph~~				
~~200~~	~~M~~	~~M~~	~~75a~~	~~100~~
~~400~~	~~M~~	~~75a~~	~~100~~	~~125~~
~~600~~	~~M~~	~~75a~~	~~125~~	~~150~~
~~800~~	~~M~~	~~100~~	~~150~~	
~~Speed = 50 mph~~				
~~200~~	~~M~~	~~75a~~	~~100~~	~~100~~
~~400~~	~~M~~	~~75a~~	~~100~~	~~125~~
~~600~~	~~M~~	~~100~~	~~125~~	~~150~~
~~800~~	~~75a~~	~~100~~	~~150~~	
~~Speed = 60 mph~~				
~~200~~	~~M~~	~~75~~	~~100~~	~~125~~
~~400~~	~~M~~	~~75~~	~~100~~	~~150~~
~~600~~	~~M~~	~~100~~	~~150~~	~~175~~
~~800~~	~~75a~~	~~125~~	~~175~~	

~~Note:~~
~~1. AASHTO (6) as adapted from Harmelink (7)~~
~~2. Table values are for left-turn volumes that are 10% of the advancing volume.~~
~~3. Queue storage lengths rounded to 25-ft intervals are not highly sensitive to the percentage of left turns. With 20% of the advancing volume turning left, the minimum left-turn storage would typically be 25 ft (one car length, including the gap between vehicles) longer than the table volumes when the sum of advancing and opposing volumes is less than 1200 veh/hr.~~
~~4. M indicates that a 50-ft minimum storage in rural areas and a 100-ft minimum in suburban/urban areas would be required.~~
~~75a = ≥ 75-ft storage required in rural areas; a 100-ft minimum should be used in suburban/urban areas.~~

Turn Bay Length

The design length of a turn bay may be controlled by either off-peak conditions or peak period conditions. Both need to be calculated, with the longer of the two generally the more desirable to be used as the design length. The following steps are suggested for determining the design length.

Determine the length for the peak period as follows:

a. Obtain the maneuver distance from Table 10-2.

b. Estimate the queue storage length:

1. For a signalized intersection use Figure 10-7 or Equation 1.
2. For an unsignalized intersection on an undivided, multilane roadway ~~use Figure 10-8 or Equation 1 using 30 intervals of 2 min per hour~~ Table B-3.
3. For an unsignalized intersection on a two-lane roadway use ~~curves such as that developed by Harmelink (7) or Table 10-5 (interpolating if necessary) or by Equation 1 using 30 intervals of 2 min per hour.~~ Table B-4.

c. Add the maneuver and the storage length to determine the total turn bay length.

1. Repeat Step b(1) for off-peak conditions (speed and volumes).

2. Obtain the minimum turn bay length by selecting the longer of the peak or off-peak length, including taper.
3. Estimate the queue length in the through-traffic lanes. Typically, this needs to be done only for the peak period.

If the queue in the through lane [Step 6 c(3)] is longer than that from Step 5 c(2), entry into the turn bay will be blocked. For vehicles to enter the turn bay under these conditions, the length of the full width of the auxiliary lane would need to be longer than the longest queue in the through traffic lane.

For example:

- Off-peak conditions
  – 40-mph progression speed
  – 60-s cycle, 60 cycles per hour
  – 85-veh/hr left turn
  – 85 veh/hr/60 cycles per hour = 1.42 vehicles per cycle
- Off-peak bay length
  – Maneuver (Table 10-2, 275 ft)
  – Queue storage = (1.42 vehicles per cycle ×2 × 25 ft) =71 ft; use 75 ft
  – Turn bay length = 275 + 75 = 350 ft
- Peak-period conditions
  – 30 mph
  – 120-s cycle, 30 cycles per hour
  – 220-veh/hr left turn
  – 220 veh/hr/ 30 cycles per hour = 7.33 vehicles per cycle
- Peak-period bay length
  – Maneuver (Table 10-2) = 160 ft
  – Queue storage = (7.33 veh/hr × 2 × 25 ft) = 366.5 ft; use 370 ft
  – Turn bay length = 160 +370 = 530 ft
- Design length of turn bays, including taper, is 530 ft (peak-period requirement of 530 ft is longer than off-peak requirement of 350 ft).

Additional Resources

J.A. Bonneson and M.D. Fontaine (2001). *Engineering Study Guide for Evaluating Intersection Improvements. NCHRP Report 457.* Transportation Research Board, Washington, D.C. http://onlinepubs.trb.org/onlinepubs/nchrp/esg/esg.pdf. Accessed August 2, 2010.

K. Fitzpatrick, M. A. Brewer, J. S. Gluck, W.L. Eisele, Y. Zhang, H. S. Levinson, W.von Zharen, M. R. Lorenz, V. Iragavarapu, E. S. Park. (2010). *Development of Left-Turn Lane Warrants for Unsignalized Intersections. NCHRP Report [to be determined].* Transportation Research Board, Washington, D.C.

ACCESS MANAGEMENT MANUAL (2003), APPENDIX A, ACCESS MANAGEMENT TECHNIQUES (PAGES 294 TO 297)

TURN BAYS AND TURN LANES

Isolated Left-Turn Bay on Undivided Roadways

Description

The isolated left-turn bay provides an auxiliary lane to remove left-turning vehicles from the through-traffic lane on an undivided roadway (Figure A-7).

Applications

The technique can be applied to

- Two-lane and undivided four-lane roads,
- High-speed roadways,
- Areas with a high left-turn volume, and
- Areas with low left-turn volume and high through volume., and
- Signalized and unsignalized locations.

Special Considerations

- The purpose of the isolated left-turn bay is to provide a protected area for left turning vehicles, thereby increasing intersection safety and reducing delay for through traffic.
- Painted channelization is commonly practiced on high-speed roads.
- Reflectorized pavement buttons often supplement the painted channelization to improve nighttime visibility.
- It is desirable to retain the shoulder wherever possible, especially on higher speed (> 45 mph) roadways.
- Lane width. The width of the left-turn lane is commonly the same as that of a through-traffic lane [e.g., 12 ft (3.6 m)] (*1, 2*). Narrower lanes may be appropriate on low-speed roadways where right-of-way is limited. The separator between the left-turn lane and the opposing traffic lane commonly consists of double yellow 4-in. (100-mm) lines. Depending on roadway width, the painted separator may range from about 1 ft (300 mm) to 4 ft (1.2 m). Where a raised separator is used, visibility and aesthetics can be enhanced by using a width that is adequate for appropriate landscaping. At its widest point, the median width is the sum of the width of the separation outside edge to outside edge) plus the width of the left-turn lane.
- At a four-way intersection, the lane may need to be sufficiently wide to store a crossing vehicle clear of both traffic streams. A width of 30 ft (9.1 m) has been suggested for design for passenger cars (*3, 4*). This generally ensures that passenger car drivers will stop within the median area and clear the edge of the traffic lanes in front and behind the stopped vehicle. In cases where adequate width cannot be provided, alternatives may be used to offset the crossroad connections so that a crossing maneuver is made by a right turn followed by a left turn or to design the turn bays as a directional opening for opposing left turns from the major roadway only.

- Bay length. The redirection taper length (L1) allows drivers to make a gradual lateral movement. The redirection taper ratio is a function of speed and may range from 15:1 at speeds less than 30 mph to 70:1 or more at 70 mph (Figure A-8).
- A short bay taper length (L2) helps to clearly identify the beginning of an auxiliary lane. The turn bay length (L4) is the sum of the bay taper length (L2) and the length of the full-width segment (L3). This length permits the driver of a left-turning vehicle to leave the through traffic lane at an acceptable speed differential with follow-through traffic and comfortably brake to a stop. This length also permits storage of vehicles waiting to complete the left turn. In developed areas, storage for four vehicles is often considered to be the minimum; storage for two vehicles is considered adequate for rural locations (*4*, Chapter 3 addresses issues in detail design of left-turn lanes, and course notes and presentation comprehensive summaries of various practices are always included). Where speed or turn volume varies throughout the day, the condition that results is the longest sum of maneuver distance plus storage. This will be the minimum bay length. See Chapter 8 for more information on the functional intersection area; Chapter 10 addresses turn bay length.

Advantages

- The left-turning vehicle is able to clear the through traffic lane at an acceptable speed.
- Rear-end and left-turn collisions are greatly reduced (*4*–*7*).
- Crash rates may be reduced by 25% (*4*).
- Capacity is increased (*4*).

Disadvantages

- The technique may require reconstruction of a considerable length of roadway to attain the additional pavement width.
- Achieving the turn bay by paint striping only results in loss of the shoulder.
- A transition by through traffic is required.

Examples

- NCHRP Project 03-91 produced guidance to identify whether a left-turn lane should be provided at an unsignalized location. See Chapter 10 for more information and the warrants (reference NCHRP Project 03-91).
- Colorado DOT has established that a warrant (*8*) for a left-turn deceleration lane is met when the peak-hour ingress volume exceeds that in Table A-4. See Chapter 5 for a description of the Colorado DOT access management categories.
- Oregon DOT criteria (*1*) are shown in Table A-5.
- ~~Many jurisdictions use the AASHTO (*9*) guidelines for left-turn lanes. These guidelines are a simplified tabular version of Harmelink's curves (*6*). Recent investigators have shown that the time drivers take to execute a left turn is longer than that assumed by Harmelink. Thus, volume warrants for left-turn bays would be less than that indicated by Harmelink's curves, and the storage length would be longer.~~
- Delaware uses a more extensive table for opposing, advancing, and left-turn volumes.

- Colorado redirection taper ratios (L1) (*8*) are

Posted Speed (mph)	*Taper Ratio*
30	15:1
40	30:1
45	45:1
50	50:1
60	60:1
70	70:1

- See Figure A-9 for examples from TX-105 in Grimes County and US-97 near Redmond, Oregon.

References

1. Standard Drawing TM539. Oregon Department of Transportation, Salem, May 2001.

2. *Design Manual—Roadway.* New Jersey Department of Transportation, Trenton, 2002. www.state.nj.us/transportation/cpm/RoadwayDesignManualMetric/interactivedownload.htm.

3. Stover, V.G., and F.J. Koepke. *Transportation and Land Development,* 2nd ed. Institute of Transportation Engineers, Washington, D.C., 2002.

4. S&K Transportation Consultants, Inc. *Access Management, Location and Design.* Participant notebook for NHI Course 133078. National Highway Institute, FHWA, Arlington, Virginia, April 2000.

5. Agent, K.R. Warrants for Left-Turn Lanes. *Transportation Quarterly,* Eno Transportation Foundation, Washington, D.C., Jan. 1983.

6. Harmelink, M.D. Volume Warrants for Left-Turn Storage Lanes at Unsignalized Grade Intersections. In *Highway Research Record 211,* HRB, National Research Council, Washington, D.C., 1967, pp. 1–18.

7. Pline, J.L. *NCHRP Synthesis of Highway Practice 225: Left-Turn Treatments at Intersections.*TRB, National Research Council, Washington, D.C., 1996.

8. *State Highway Access Code.* Colorado Department of Transportation, Denver, effective Aug. 31, 1998.
www.dot.state.co.us/BusinessCenter/Permits/Access/601_1_AccessCode_May2002.pdf.

9. *A Policy on Geometric Design of Highways and Streets.* AASHTO, Washington, D.C., 1984, 1990, 1994.

Additional Resources

Left-Turn Lane Criteria. Oregon Department of Transportation, Salem, June 22, 1996.

Guidelines for Left-Turn Lanes. Committee 4A-22 Final Report. Institute of Transportation Engineers, Washington, D.C., Sept. 1991.

Gluck, J., H.S. Levinson, and V. Stover. *NCHRP Report 420: Impacts of Access Management Techniques*. TRB, National Research Council, Washington, D.C., 1999.

Geometric Design Standards. Ontario Ministry of Transportation, Toronto, Canada.

ACCESS MANAGEMENT MANUAL (2003), APPENDIX A, ACCESS MANAGEMENT TECHNIQUES, TURN BAYS AND TURN LANES (PAGES 298 TO 300)

Shoulder Bypass at Three-Way (T) Intersection

Description

The shoulder bypass allows a through vehicle to bypass a left-turning vehicle that is stopped in the traffic lane (Figure A-10).

Applications

The technique can be applied

- To three-way intersections on minor roadways,
- To locations where space is not available for an isolated left-turn bay, and
- As a temporary solution until an isolated left-turn bay can be constructed.

Special Considerations

- The technique is appropriate for locations with low left-turn volumes.
- It is also appropriate for low-volume roadways.
- The bypass lane is typically the same width as a regular traffic lane.
- Approach and departure tapers are typically much shorter than the redirection tapers used for isolated left-turn bays.
- A narrow shoulder of earth and grass is commonly provided on the outside of the bypass lane.

Advantages

- Safety is improved, because the bypass helps reduce the potential for rear end collisions.
- Delays to through traffic are reduced. It is inexpensive to implement, especially if a paved shoulder >10 ft wide already exists.
- The bypass takes less space than an isolated left-turn bay.

Disadvantages

- The bypass requires through traffic to transition around vehicles stopped in the through lane.
- It is less safe than an isolated left-turn bay.
- Driver expectancy is violated.
- Earthwork may be necessary to add the bypass. Additional right-of-way may be needed for the widening and for modifying the drainage.

Examples

- NCHRP Project 03-91 produced guidance to identify whether a left-turn shoulder bypass should be provided at an unsignalized location on rural two-lane and four-lane roadways. See Chapter 10 for more information and the warrants (reference NCHRP Project 03-91).
- In Becker County, Minnesota, shoulder bypasses are used for T-intersections for major roads with an annual average daily traffic (AADT) > 500 and for minor roads with an AADT > 250 (D. Heyer, Detroit Lakes, Minnesota, personal communication, Feb. 22, 1999).
- In Minnesota DOT warrants, the technique is applied for T-intersections that do not meet left-turn warrants (*1,* Section 5-3-01.02; *2*).
  The bypass is designed with a 54-m-long approach and departure tapers and a 75-m full-width section of which 15 m are beyond the far edge of the intersecting road.
- Connecticut DOT uses a shoulder bypass lane where space is inadequate for its standard left-turn lane design (discussions with Minnesota DOT personnel during National Highway Institute Course 133078, April 2000).
- The Texas DOT uses the shoulder bypass on a case-by-case basis. However, there are no specific warrants for the technique (correspondence with Texas DOT District 21 traffic engineer).
- Delaware DOT warrants (*2*) are as follows:

Volume (AADT)		
Highway	*Left-Turn*	*Technique Applied*
<2,000	>200	Bypass lane
2,001–4,000	201–400	Bypass lane
	>400	Consider left-turn lane
>4,000	50–400	Bypass lane
	>400	Consider left-turn lane

- For Michigan DOT warrants, see Figure A-11 (*2*).
- The schema for Michigan DOT design is shown in Figure A-12.
- Figure A-13 shows an example of the technique used at an intersection with a road serving a resort and farms in Becker County, Minnesota. Speeds on the road exceed 60 mph.

References

1. *Road Design Manual.* Minnesota Department of Transportation, St. Paul, Minnesota, Dec. 24, 1996.

2. *Note on Warrant for Passing Flares at Driveways.* Traffic and Safety Division, Michigan Department of Transportation, Lansing, Michigan, Nov. 30, 1988.

ACCESS MANAGEMENT MANUAL, APPENDIX A, ACCESS MANAGEMENT TECHNIQUES, TURN BAYS AND TURN LANES (PAGES 303 TO 304)

Left-Turn Bay at Median Opening

Description

The left-turn bay at a median opening provides for left-turn deceleration and storage at a signalized or an unsignalized median opening (Figure A-16).

Application

It can be applied to all median openings, both signalized and unsignalized.

Special Considerations

- Issues involving design of a left-turn bay (such as taper, maneuver distance, and storage) are addressed in Chapter 10. See also *Access Management, Location and Design* for warrants and guidelines for left-turn bays (*1*, Chapter 3) and for design guidelines (*1*, Chapter 4).
- The width of the left-turn lane and the divider separating the left turns from the opposing traffic will depend upon the available median width. Below are examples of the combinations of lane and separation widths that might be considered for different median widths, as adapted from the National Highway Institute (*1*):

Median Width (ft)	*Left-Turn Lane Width (ft)*	*Divider*
16	11 or 12	4 ft raised
14	11	3 ft raised
14	10	4 ft raised
12	11	4 in double yellow lines
12	10	2 ft slightly raised

- In urban and suburban areas, a short taper does not restrict lateral movement into the left-turn lane at the slow speeds encountered during peak periods.
- Short tapers on horizontal curves to the left help keep drivers of through vehicles from inadvertently entering the left-turn bay.
- A turn bay of appropriate length allows left-turning vehicles to clear the through lane without interfering unduly with following through traffic and then decelerate to a stop before reaching the end of a queue of vehicles waiting to complete the left turn.
- Left-turn lanes are normally the same width as a traffic lane. However, lanes as narrow as 9 ft (2.75 m) have been used in urban areas where right-of-way is limited and speeds are very slow [e.g., ≤30 mph (≤50 km/h)].

Advantages

- The technique provides refuge for drivers who are making left turns, thereby minimizing impacts on through traffic (*2*).
- It allows left-turning vehicles to leave the through lane with an acceptable speed differential with through traffic (*1, 3*).
- Crash rates at unsignalized median openings may be reduced by 50% to more than 75% (*2–6*) and by about 20% to more than 55% at signalized locations (*1*, Chapter 5; *2; 4*).

- Capacity can be increased by 25% or more (*1*, Chapter 2; *2; 5*).
- Delay to through traffic is reduced (*2*).

Disadvantages

- The left-turn bay cannot be used on medians that are too narrow to provide a left-turn lane and a separator between the left-turn lane and the opposing traffic lane.
- Proximity of the bay to any other median openings may limit the length of the turn lane or require closure of a less important opening.

Examples

- NCHRP Project 03-91 produced guidance to identify whether a left-turn lane should be provided at an unsignalized location. See Chapter 10 for more information and the warrants (reference NCHRP Project 03-91).
- Vancouver, British Columbia, in Canada has a program to provide existing intersections with left-turn bays.
- The DOTs of Colorado, Florida, and Oregon call for left-turn bays at all median openings on divided highways.
- Florida DOT practice is to add turn bays as part of resurfacing projects. In some resurfacing projects, the Florida DOT also converts unsignalized full median openings in suburban and urban areas to directional left-turn and U-turn openings.

References

1. S&K Transportation Consultants, Inc. *Access Management, Location and Design.* Participant notebook for NHI Course 133078. National Highway Institute, FHWA, Arlington, Virginia, April 2000.

2. Gluck, J., H.S. Levinson, and V. Stover. *NCHRP Report 420: Impacts of Access Management Techniques*. TRB, National Research Council, Washington, D.C., 1999.

3. Stover, V.G., and F.J. Koepke. *Transportation and Land Development,* 2nd ed. Institute of Transportation Engineers, Washington, D.C., 2002.

4. Hagenauer, G.F., J. Upchurch, D. Warren, and M.J. Rosenbaum. Intersections. In *Synthesis of Safety Research Related to Traffic Control and Roadway Elements,* Vol. 1, Report FHWA-TS-82-232, FHWA, U.S. Department of Transportation, Dec. 1982.

5. Rudberg, D.H. Arterial Streets: Important Components of Vancouver's Urban Transportation System. *ITE Journal*, Oct. 1988.

6. Wilson, J.E. Simple Types of Intersection Improvements. In *Special Report 93: Improved Street Utilization through Traffic Engineering,* Highway Research Board, National Research Council, Washington, D.C., 1967.

Additional Resources

A Policy on Geometric Design of Highways and Streets. AASHTO, Washington, D.C., 1984, 1990, 1994.

Oppenlander, J.C., and J.E. Oppenlander. Storage Requirements for Signalized Intersection Approaches. *ITE Journal,* Feb. 1996.

Oppenlander, J.C., and J.E. Oppenlander. Complete Tables for Storage Requirements for Signalized Intersection Approaches. Supplement to *ITE Journal,* Feb. 1996.

Oppenlander, J.C., and J.E. Oppenlander. Storage Lengths for Left-Turn Lanes with Separate Phase Control. *ITE Journal,* July 1994.

Koepke, F.J., and H.S. Levinson. *NCHRP Report 348: Access Management Guidelines for Activity Centers.* TRB, National Research Council, Washington, D.C., 1992.

APPENDIX C

STATE WARRANTS/GUIDELINES FOR LEFT-TURN LANES

State	Material on Left-Turn Guidelines or Warrants
Alabama	**Currently offline**
Alaska http://www.dot.state.ak.us/stwddes/dcsprecon/assets/pdf/preconhwy/ch11/chapter11.pdf *Alaska Highway Preconstruction Manual* January 2005	**1190. Driveway Standards** **1190.5 Control Dimensions** 11. **Speed Change Lane and Left-Turn Lanes:** On high-speed (50 mph or over) or high-volume arterial roadways, speed change lanes may be required for the acceleration or deceleration of vehicles entering or leaving the public roadway from or to a higher-volume traffic generation (greater than or equal to 100 vehicles per hour) or attracting development. Use Figure 4-3 of NCHRP 279 Intersection Channelization Design Guide as a guideline for the right-turn treatments. On a one-way street, the above criteria also apply to the left through lane. For guidelines on the need for left-turn lanes on a main street or road at a driveway, refer to Exhibit 9-75 in AASHTO *A Policy on the Geometric Design of Highways and Streets 2001*. **1150.2. Urban Arterials** ***1150.2.1. General Design Considerations*** Design of urban arterials shall conform to recommendations in the *Green Book*. ***1150.2.2. Medians*** Median openings generally permit cross traffic and left turns which conflict with the through traffic on the arterial. These conflicts result in delays and accident exposure, which you can minimize by providing as few median openings as possible. However, keep in mind that restriction of mid-block left turns often substantially increases the number of U-turns at the adjacent intersection with median openings. A similar situation exists where a minor street intersects the arterial and does not provide a median opening. Generally, provide median openings only if the volume of cross- or left-turn traffic is relatively large, such as at another arterial or major collector street or, in some cases, at an access point to a major traffic generator, such as a regional shopping center or industrial plant. Because the openings are at major traffic points, assume that at some time, if not immediately, these median opening locations will be signalized. Additionally, where signalized intersections are 0.5 mile or less apart, efficiency and safety require interconnected or synchronized signals to achieve smooth traffic flow along the arterial.
Arizona http://www.azdot.gov/highways/Roadway_Engineering/Roadway_Design/Guidelines/Manuals/PDF/RoadwayDesignGuidelines.pdf *Roadway Design Guidelines* July 2008	**408.10 — Left-turn Channelization** *A) General:* A left-turn lane serves to expedite the flow of through traffic, to control the movement of turning traffic, and to improve the safety and capacity of the intersection. Traffic Engineering Group will analyze the traffic movements and other factors at an intersection to determine the need for a separate left-turn lane(s) and establish the vehicle storage requirements for the lane(s).

State	**Material on Left-Turn Guidelines or Warrants**
Arkansas	**Not available online**
California http://www.dot.ca.gov/hq/oppd/hdm/pdf/english/chp0400.pdf *Highway Design Manual* Chapter 400: Intersections at Grade July 2009	**405.2 Left-turn Channelization** *(1) General.* The purpose of a left-turn lane is to expedite the movement of through traffic, control the movement of turning traffic, increase the capacity of the intersection, and improve safety characteristics. The District Traffic Branch normally establishes the need for left-turn lanes. See "Guidelines for Reconstruction of Intersections," August 1985, published by the California Division of Transportation Operations.
Colorado http://www.dot.state.co.us/DesignSupport/Design%20Guide%2005/DG05%20Ch%2009%20Intersections.pdf http://www.dot.state.co.us/DesignSupport/Design%20Guide%2005/Index.htm *Design Guide* Chapter 9—Intersections April 2006	**9.18.6.1 Median Left-Turn Lane Warrants** To facilitate flow where the intersection is unsignalized, the following guidelines are suggested: • Left-turn lanes should be considered at all median crossovers on divided, high-speed highways. • Left-turn lanes should be provided at all uncontrolled approaches of primary, high-speed rural highway intersections with other arterials and collectors. • Left-turn lanes should be provided on stopped or secondary approaches based on analysis of the capacity and operations of the unsignalized intersection.
Connecticut http://www.conndot.ct.gov/publications/hdm/Chapter%2011.pdf *ConnDOT Highway Design Manual* Chapter 11: Intersections At-Grade December 2006	In general, exclusive left-turn lanes should be provided for at-grade intersections as follows: 1. on all divided urban and rural highways with a median wide enough to allow a left-turn lane (this applies to intersections with public roads and to major traffic generators); 2. for all approaches at arterial/arterial intersection; 3. at any unsignalized intersection on a two-lane urban or rural highway which satisfies the criteria in Figures 11-5B, 11-5C, 11-5D, 11-5E or 11-5F **[appears to be based on *NCHRP Report 279*/Harmelink, also includes graphs for 55 and 45 mph, and does *not* include the four-lane undivided graph]**; 4. at any intersection where a capacity analysis determines a left-turn lane is necessary to meet the level-of-service criteria; 5. at any intersection included or expected to be within an interconnected signal system where the presence of standing left-turning vehicles would disrupt the progression of platooned traffic; or 6. at any intersection where the crash experience, traffic operations, sight distance restrictions or engineering judgment indicates a significant problem related to left-turning vehicles.

State	Material on Left-Turn Guidelines or Warrants
Delaware http://www.deldot.gov/information/pubs_forms/manuals/road_design/pdf/07_intersections.pdf *DelDOT Road Design Manual* Chapter 7—Intersections November 2006	For unsignalized intersections, left-turn lanes should be provided: • At all median openings on high-speed divided highways. • On approaches where sight distance is limited. • At non-stopping approaches of rural arterials and collectors. • At other approaches where required based on capacity and operational analysis. There may be other needs, primarily safety, for left-turn lanes at other locations than mentioned in these general guidelines. For two-lane roadways, a high-volume, intersecting minor roadway or entrance may also create the need to separate movements with auxiliary turning lanes. Figure 7-12 **[*Green Book* values]** is a tabulated guide to traffic volumes where left-turn lanes should be considered on two-lane highways. For the values shown, left turns and right turns from the minor street can be equal to, but not greater than, the left turns from the major street.
Florida http://www.dot.state.fl.us/rddesign/FloridaGreenbook/2007/2007FloridaGreenbook.pdf *Manual on Uniform Minimum Standards for Design, Construction, and Maintenance for Streets and Highways*	**Did not find discussion on left-turn installation guidelines or warrants**
Florida http://www.dot.state.fl.us/planning/systems/sm/accman/pdfs/driveway2008.pdf *Driveway Information Guide* September 2008	**Section 3.6: Left Turn Lanes Serving Driveways on Multilane and 2 Lane Roadways** On a multilane roadway with a median Whenever a driveway is directly served by a median opening, a left turn lane should be available. This provides for the safest left turns into the driveway. On a two-lane roadway Exclusive left turn lanes should be considered at any location serving the public, especially on curves and where speeds are 45 mph and higher. The AASHTO Green Book contains guidance on this issue. However, the guidelines were developed based on delay rather than crash avoidance. Safety is the main reason behind exclusive left turn lanes.

<table>
<tr><th>State</th><th>Material on Left-Turn Guidelines or Warrants</th></tr>
<tr><td>Georgia

http://www.dot.ga.gov/doingbusiness/PoliciesManuals/roads/Documents/DesignPolicies/DrivewayFull.pdf

Regulations for Driveway and Encroachment Control

March 2004</td><td>4I-1-2 Minimum Requirements for Left Turn Lanes
Left turn lanes must be constructed at no cost to the Department if the daily site generated Left Turn Volumes (LTV) based on ITE Trip Generation (assuming a reasonable distribution of entry volumes) meet or exceed the values shown in Table 4-7a Condition 1. If the LTVs are below the requirements for Condition 1, the applicant may be required to construct a Right Hand Passing Lane (see Appendix F) if they meet the criteria in Table 4-7b Condition 2. The District Access Management Engineer will use engineering judgment to determine if the field conditions would allow construction of the Right Hand Passing Lane. Passing lane sections fall under the criteria for two or more lanes.

Table 4-7a. Georgia's Regulations for Driveway and Encroachment Control Left Turn Requirements, Condition 1.
<table>
<tr><th colspan="5">Condition 1: Left Turn Requirements—Full Construction</th></tr>
<tr><th rowspan="2">Posted Speed</th><th colspan="2">2 Lane Routes</th><th colspan="2">More than 2 Lanes on Main Road</th></tr>
<tr><th><6000 ADT</th><th>≥6000 ADT</th><th><10000 ADT</th><th>≥10000 ADT</th></tr>
<tr><td>35 mph or less</td><td>300 *</td><td>200 *</td><td>400 *</td><td>300 *</td></tr>
<tr><td>40 to 50 mph</td><td>250 *</td><td>175 *</td><td>325 *</td><td>250 *</td></tr>
<tr><td>≥55 mph</td><td>200 *</td><td>150 *</td><td>250 *</td><td>200 *</td></tr>
<tr><td colspan="5">* LTV a day</td></tr>
</table>

Table 4-7b. Georgia's Regulations for Driveway and Encroachment Control Left Turn Requirements, Condition 2.
<table>
<tr><th colspan="3">Condition 2: Left Turn Requirements w/R□ght Hand Passing L□ne Option</th></tr>
<tr><th rowspan="2">Posted Speed</th><th colspan="2">2 Lane Routes Only</th></tr>
<tr><th><4000 ADT</th><th>≥4000□ADT</th></tr>
<tr><td>35 mph or less</td><td>200 LTV a day</td><td>125 LTV a day</td></tr>
<tr><td>40 to 50 mph</td><td>100 LTV a day</td><td>75 LTV a day</td></tr>
<tr><td>≥55 mph</td><td>75 LTV a day</td><td>50 LTV a day</td></tr>
</table>

In the event the District Access Management Engineer determines that field conditions or other factors indicate that it would be in the best interest of the Department to waive the left turn lane requirement, the District Access Management Engineer must document the recommendations using the form in Appendix E. The recommendations shall be approved by the District Engineer and be attached to the Permit. The District Access Management Engineer may also require the addition of a Left Turn lane, even when the conditions in Table 4-7 are not met, if roadway geometry or field conditions indicate that the safety of the traveling public would be improved. The recommendation must be documented and approved by the District Engineer for inclusion with the Permit.</td></tr>
<tr><td>Hawaii

http://hawaii.gov/dot/highways</td><td>Did not find a link for a design manual type of document</td></tr>
<tr><td>Idaho

http://itd.idaho.gov/manuals/Online_Manuals/Design/Design_Manual.htm

Roadway Design Manual

July 2008</td><td>Did not find information on left-turn lane warrants</td></tr>
</table>

State	Material on Left-Turn Guidelines or Warrants
Illinois http://dot.state.il.us/desenv/BDE%20Manual/BDE/pdf/chap36.pdf *Bureau of Design and Environment Manual* Chapter 36—Intersections December 2002	**36-3.01(b) Left-Turn Lanes** The accommodation of left turns is often the critical factor in proper intersection design. Left-turn lanes can significantly improve both the level of service and intersection safety. Always use an exclusive left-turn lane at all intersections on divided urban and rural highways with a median wide enough to accommodate a left-turn lane, regardless of traffic volumes. Consider using an exclusive left-turn lane for the following: • at any unsignalized intersection on a two-lane urban or rural highway which satisfies the criteria in Figures 36-3C, D, E, F, or G **[appears to be based on Harmelink/*NCHRP Report 279*]**; • at any signalized intersection where the left-turning volume is equal to or greater than 75 veh/hr for a single turn lane or 300 veh/hr for a dual turn lane; • any intersection where a capacity analysis determines a left-turn lane is necessary to meet the level-of-service criteria, including dual left-turn lanes; • for uniformity of intersection design along the highway if other intersections have left-turn lanes (i.e., to satisfy driver expectancy); or • any intersection where the crash experience, traffic operations, sight distance restrictions (e.g., intersection beyond a crest vertical curve), or engineering judgment indicates a significant conflict related to left-turning vehicles.

<table>
<tr><th>State</th><th>Material on Left-Turn Guidelines or Warrants</th></tr>
<tr><td>Indiana

http://www.state.in.us/dot/div/contracts/standards/dm/english/Part5Vol1/ECh46/ch46.htm

Indiana Design Manual, Road Design, Intersections At-Grade

December 3, 2008

Document Revised January 2010. Updated URL: http://www.in.gov/dot/div/contracts/standards/dm/Part5/Ch46/ch46.htm</td><td>The Indiana Design Manual provides information on warrants for left-turn lanes (Section 46-4.01(02)) and passing blisters (Section 46-4.03). An exclusive left-turn lane should be provided as follows:
at each intersection on an arterial, where practical;
at each intersection on a divided urban or rural highway with a median wide enough to accommodate a left-turn lane, provided that adequate spacing exists between intersections;
at an unsignalized intersection on a two-lane urban or rural highway which satisfies the criteria shown in [a table that contains the Green Book values];
at an intersection where a capacity analysis determines that a left-turn lane is necessary to meet the level-of-service criteria, including multiple left-turn lanes;
at a signalized intersection where the design-hour left-turning volume is 60 veh/h or more for a single turn lane, or where a capacity analysis determines the need for a left-turn lane;
for uniformity of intersection design along the highway if other intersections have left-turn lanes in order to satisfy driver expectancy;
at an intersection where the accident experience, traffic operations, sight distance restrictions (e.g., intersection beyond a crest vertical curve), or engineering judgment indicates a significant conflict related to left-turning vehicles; or
at a median opening where there is a high volume of left turns, or where vehicular speeds are 50 mph or higher.
The Indiana Design Manual provides information on warrants for passing blisters. Passing blisters are used to relieve congestion due to left-turning vehicles. Appendix B includes a copy of a layout for a passing blister. Indiana states the following is to be reviewed to determine the need for a passing blister:
Traffic volume. A passing blister may be provided at the intersection of a public road or street with a two-lane state highway with a design-year AADT of 5000 or greater. For a two-lane state highway with a design-year of less than 5000, a passing blister should be used only if one or more of the following occurs:
There is an existing passing blister.
There are 20 or more left-turning vehicles during the design hour.
Accident reports or site evidence, such as skid marks in the through lane displaying emergency braking, indicate potential problems with left-turning vehicles.
The shoulder indicates heavy use (e.g., dropped shoulder, severe pavement distress).
The decision on whether to use either a channelized left-turn lane or a passing blister should be based on accident history, right-of-way availability, through- and turning-traffic volumes, design speed, and available sight distance. A channelized left-turn lane should be provided if the left-turning volume is high enough that a left-turn lane is warranted.</td></tr>
</table>

State	Material on Left-Turn Guidelines or Warrants
Iowa ftp://165.206.203.34/design/dmanual/00_START HERE_TOC.pdf updated URL: http://www.iowadot.gov/design/dmanual/manual.html *Design Manual,* Section 6C-5 November 25, 2008	**Left Turn Lanes** Left turn lanes provide storage in the median for left-turning vehicles, or when warranted, deceleration outside of the through traffic lanes for left-turning vehicles. All Type "A" and high volume Type "B" entrances should have left turn lanes provided, see Section 3E-2 of this manual. If a left turn deceleration is not warranted, a left turn storage lane should be provided. Normally, left turn lanes are designed as parallel lanes. **Left Turn Deceleration Lane Warrants** The basic guidelines for when left turn deceleration lanes are warranted involve mainline turning and approach volume, and intersection location. **Turning and approach volume** A left turn deceleration lane may be warranted if left turning traffic flow rate is greater than 30 vehicles per hour measured over a minimum of 15 minutes and either: a. approach volume is greater than 400 vehicles per hour, or b. approach truck traffic volume is greater than 40 vehicles per hour. **Intersection location** Intersection location may warrant a left turn deceleration lane even if turning and approach volumes do not. To improve operational efficiency, left turn deceleration lanes should be considered for intersections located within approximately 5 miles (8 kilometers) of an urban area with a population of 20,000 or greater. Other locations where left turn deceleration lanes may be judged to be warranted by the PMT include schools, main entrances for towns, shopping areas, housing developments, attraction locations such as recreational areas, and locations that would have special users such as truck traffic or campers. Special attention should be given to intersections serving locations that attract elderly drivers such as drug stores, grocery stores, retirement developments, medical facilities, nursing homes, etc. **Entrance Types** **Type "A" entrance**. An entrance developed to carry sporadic or continuous heavy concentrations of traffic. Generally, a Type "A" entrance carries in excess of 150 vehicles per hour. An entrance of this type would normally consist of multiple approach lanes and may incorporate a median. Possible examples include racetracks, large industrial plants, shopping centers, subdivisions, or amusement parks. **Type "B" entrance**. An entrance developed to serve moderate traffic volumes. Generally, a Type "B" entrance carries at least 20 vehicles per hour but less than 150 vehicles per hour. An entrance of this type would normally consist of one inbound and one outbound traffic lane. Possible examples include service stations, small businesses, drive-in banks, or light industrial plants. **Type "C" entrance**. An entrance developed to serve light traffic volumes. Generally, a Type "C" entrance carries less than 20 vehicles per hour. An entrance of this type would not normally accommodate simultaneous inbound and outbound vehicles. Possible examples include residential, farm or field entrances.
Kansas	**Not available online**

Kentucky
 	
http://transportation.ky.gov/design/designmanual/chapters/12Chapter%200900%20AS%20PRINTED%202006.pdf
 	
KYTC Highway Design,
Intersection Chapter
 	
January 2006
	Unsignalized Intersection:
	• Left-turn lanes should be provided at:
	  o Median openings on divided roadways but not at median crossovers on freeways and interstates
	  o All non-stopping approaches of rural arterials and collectors
	  o All other approaches where required on the basis of capacity, safety, and operational analysis
	• Left-turn lanes should be considered where sight distance is limited.

State	Material on Left-Turn Guidelines or Warrants
Louisiana http://www.dotd.louisiana.gov/highways/project_devel/design/road_design/road_design_manual/Road_Design_Manual_(Full_Text).pdf *Roadway Design Procedures and Details* July 2002 Document updated January 2009. Updated URL: http://www.dotd.louisiana.gov/highways/project_devel/design/road_design/documents.aspx	On new four-lane highways, left turn lanes are usually provided at all intersecting side roads that have dedicated right-of-way. Right turn lanes, left turn lanes on divided roadways (both depressed and raised median sections), and dedicated left turn lanes on five-lane urban roadways are considered based on: __ traffic volumes __ turning movements __ reduced accident potential __ and increased operational efficiency
Maine	**Did not find any discussion on left-turn warrants**
Maryland http://www.sha.state.md.us/businesswithsha/bizStdsSpecs.asp?id=B157+B159 *Book of Standards for Highway and Incidental Structures*	**Did not find any discussion on left-turn warrants**
Massachusetts http://www.vhb.com/mhdGuide/mhd_GuideBook.asp http://www.vhb.com/mhdGuide/pdf/CH_6.pdf *Highway Design Guidebook* Chapter 6: Intersection Design 2006 Edition	**6.7.3.3 General Criteria for Right-Turn and Left-Turn Lanes** Criteria for considering installation of left-turn lanes are summarized in Exhibit 6-23 **[*Green Book* values along with criteria for 30 mph]**. Considerable flexibility should be exercised in considering left-turn lanes. Typically, they involve little impact to the setting, while generally yielding large benefits in safety and user convenience. Left-turn lanes may be desirable in many situations with volumes well below those stated. These include to destinations of special interest (shopping, major institutions, etc.), or for locations with marginal sight distance on the main road or a consistent occurrence of rear-end crashes.
Michigan http://mdotwas1.mdot.state.mi.us/public/design/englishroadmanual/ *Road Design Manual*	**Did not find any discussion on left-turn warrants**

State	Material on Left-Turn Guidelines or Warrants

Minnesota
 	
http://www.dot.state.mn.us/design/rdm/english/5e.pdf
 	
Road Design Manual Chapter 5: At-Grade Intersections
 	
June 2000
	5-3.01.01 Turn Lane Policy at Urban Intersections
	Because of the operational and safety benefits associated with right and left-turn lanes, it is Mn/DOT's policy that, in urban areas, they be considered wherever construction is economically feasible taking into account amount of right of way needed, type of terrain, and environmentally or culturally sensitive areas.
	For new construction/reconstruction projects on divided highways, left-turn and right-turn lanes should be considered at all locations where a paved crossover will be constructed.
	For preservation projects, left-turn lanes should, if feasible, be provided:
	1. At all public road median crossovers.
	2. At non-public access locations generating high traffic volumes.
	3. At locations where accident records confirm the existence of an excessive hazard.
	4. At locations determined by the District Traffic Engineer in consideration of accidents, capacity and traffic volumes.
	5. Where a median opening is planned or exists, and its continued existence is justified, a left-turn lane may be added regardless of what the access point serves.
	5-4.01 Turn Lanes
	As with urban highways, the degree of access control greatly influences the accident rate and efficiency of traffic operation on rural highways. Therefore, designers should try to close any unjustified or potentially dangerous access points. However, if an access point is to remain open, adding a turn lane will enhance the operation and safety.
	1. In addition to the policies listed below, left-turn and right-turn lanes should be constructed at locations determined by the District Traffic Engineer in consideration of accidents, capacity, and traffic volumes.
	2. Where a median opening is planned, or already exists and its continued existence is justified, a left-turn lane may be added regardless of what the access point serves.
	3. Turn lanes should be considered at every public road intersection along a stretch of highway if most intersections on the stretch meet the warrants. If most intersections have turn lanes, motorists will come to expect all intersections to have them.
	5-4.01.01 Policy on Multi-Lane Highways
	1. Right-turn and left-turn lanes should be standard features at all public access points.
	2. Right-turn and left-turn lanes are also warranted if the access point serves an industrial, commercial, or any substantial trip-generating land use, or if the access point serves more than three residential units.
	5-4.01.02 Policy on two-lane Rural Highways
	1. Right-turn lanes should be considered when the projected ADT is over 1500, the design speed is 45 mph or higher, and the following:
	  a. At all public road access points.
	  b. If industrial, commercial, or substantial trip generating land use is to be served, or
	  c. If the access serves more than 10 residential units.
	2. Left turn lanes should be provided when the access is to a public road, an industrial tract or a commercial center.
	3. The designer, in conjunction with the District Traffic Engineer, may select either a channelized or a painted left-turn lane. The selection will be based on a number of factors including accident history, traffic volume, comparative costs, availability of right of way, environmental impacts, and physical features such as sight distance.

<table>
<tr><th>State</th><th>Material on Left-Turn Guidelines or Warrants</th></tr>
<tr><td>Minnesota

http://www.oim.dot.state.mn.us/access/index.html

Access Management, Chapter 3

Updated URL:
http://www.dot.state.mn.us/accessmanagement/pdfchapters/chapter3.pdf</td>
<td>Turn lanes are to be provided at public street connections and driveways in accordance with the Mn/DOT Road Design Manual, Section 5-3, and the guidance provided in the Access Management Manual. Guidance provided includes the following:

Warrant 7: Crash History — At high-volume driveways (>100 trips per day) and all public street connections that demonstrate a history of crashes of the type suitable to correction by a turn lane or turn-lane treatment (typically three or more correctable crashes in one year), or where adequate trial of other remedies has failed to reduce the crash frequency.
Warrant 8: Corridor Crash Experience — On highway corridors that demonstrate a history of similar crash types suitable to correction by providing corridor-wide consistency in turn-lane use.
Warrant 9: Vehicular Volume Warrant — At high-volume driveways (>100 trips per day) and all public street connections on high-speed highways (posted speed ≥ 45 mph) that satisfy the criteria in Table below.

Minnesota Access Management Manual, Warrant for Left-Turn Lanes.
<table>
<tr><th>Two-Lane Highway ADT</th><th>Four-Lane Highway ADT</th><th>Cross Street or Driveway ADT</th><th>Turn Lane Requirement</th></tr>
<tr><td>1500 to 2999</td><td>3000 to 5999</td><td>> 1500</td><td>Left-turn lane warranted</td></tr>
<tr><td>3000 to 3999</td><td>6000 to 7999</td><td>> 1200</td><td>Left-turn lane warranted</td></tr>
<tr><td>4000 to 4999</td><td>8000 to 9999</td><td>>1000</td><td>Left-turn lane warranted</td></tr>
<tr><td>5000 to 6499</td><td>10,000 to 12,999</td><td>>800</td><td>Left-turn lane warranted</td></tr>
<tr><td>≥ 6500</td><td>≥ 13,000</td><td>100 to 400</td><td>Left-turn or bypass lane</td></tr>
<tr><td>≥ 6500</td><td>≥ 13,000</td><td>≥ 400</td><td>Left-turn lane warranted</td></tr>
<tr><td colspan="4">Highway AADT one year after opening.
Posted speed 45 mph or greater.</td></tr>
</table>
</td></tr>
<tr><td>Mississippi
http://www.gomdot.com/Divisions/Highways/Resources/RoadwayDesign/pdf/Manual/2001/chapter06.pdf

Roadway Design Manual, Chapter 6 At-Grade Intersections/Interchanges

2001</td>
<td>6-4.02 Warrants for Left-Turn Lanes
The accommodation of left turns is often the critical factor in proper intersection design.

Left-turn lanes influence both the level of service and intersection safety. Exclusive left turn lanes should be provided:

at all median openings (crossovers) on divided urban and rural highways with a median wide enough to accommodate a left-turn lane;
at any unsignalized intersection on a two-lane highway which satisfies the criteria in Figures 6-48, 6-4C or 6-4D [appears to be based on Harmelink/NCHRP Report 279];
at any unsignalized intersection on a four-lane undivided highway which satisfies the criteria in Figure 6-4E;
at any intersection where a capacity analysis determines a left-turn lane is necessary to meet the level-of-service criteria; or
at any intersection where the accident experience, existing traffic operations, adverse geometrics (e.g., restricted sight distance) or engineering judgment indicate a significant hazard related to left-turning vehicles.

</td></tr>
</table>

<table>
<tr><th>State</th><th>Material on Left-Turn Guidelines or Warrants</th></tr>
<tr><td>Missouri

http://epg.modot.mo.gov/index.php?title=940.9_Auxiliary_Acceleration_and_Turning_Lanes

Engineering Policy Guide

August 2007</td><td>Dedicated left- and right-turn lanes are to be provided in situations where traffic volumes and speeds are relatively high and conflicts are likely to develop at public road intersections and driveways between through and turning traffic. Auxiliary lanes are an asset in promoting safety and improved traffic flow in such situations. Some major applications and considerations for the design of auxiliary lanes are as follows:
Installing a right-turn acceleration lane. These lanes allow entering vehicles (those that have turned right from a driveway or minor public road onto the major route) to accelerate before entering the through-traffic flow. Acceleration lanes are to be considered on roadway segments, intersections and driveways with high traffic volumes where speed differential could result in unacceptable conflicts and/or delay. Acceleration lanes may also be appropriate where crash experience indicates a problem with right turning, entering vehicles. The right-turn acceleration lane is to be sufficiently long to allow safe and efficient merge maneuvers. The design length, tapers and other features of right-turn acceleration lanes are to be guided by a traffic study.
Installing auxiliary left-turn lanes. Such lanes, installed in the roadway center, are intended to remove turning vehicles from the through traffic flow. This reduces the frequency of rear-end collisions at locations where there is considerable left-turn ingress activity, such as major driveways and minor public road intersections. Left-turn lane warrants are shown in the following figures [not included]. To use the figures, peak-hour traffic counts including directional splits, which may be obtained from district Traffic staff, will be required. In addition, the ITE Trip Generation Manual may be used as an estimate for peak-hour traffic counts. Planning can provide necessary growth rates for design-year analyses.
Left-turn lane guidelines for two-lane roads less than or equal to 40, 45, 50, 55, 60 mph
Left turn lane guidelines for four-lane undivided roadways
[these figures are similar to NCHPR Report 279/Harmelink's graphs]
The use and design of auxiliary left-turn lanes are to be guided by a traffic study. In general, auxiliary left-turn lanes must be long enough to accommodate a safe deceleration distance and provide adequate storage for an expected peak-hour turning traffic queue. Refer to storage and deceleration lengths for additional information.</td></tr>
<tr><td>Montana

http://www.mdt.mt.gov/other/roaddesign/external/montana_road_design_manual/13_intersection_at-grade.pdf

http://www.mdt.mt.gov/publications/manuals.shtml

Road Design Manual, Chapter 13: Intersections At-Grade

December 2004</td><td>13.3.1.2 Guidelines for Left-Turn Lanes
Exclusive left-turn lanes should be considered:
at all public intersections on all multilane urban and rural highways, regardless of traffic volumes;
at the free-flowing leg of any unsignalized intersection on a two-lane urban or rural highway which satisfies the criteria in Figures 13.3C, 13.3D, 13.3E or 13.3F [appears to be based on Harmelink/NCHRP Report 279];
at any intersection where a capacity analysis determines a left-turn lane is necessary to meet the level-of-service criteria;
as a general rule on the major roadway, at any signalized intersection;
at high-volume driveway approaches which satisfy the criteria in Figures 13.3C, 13.3D, 13.3E or 13.3F; or
at any intersection where the accident experience, traffic operations, sight distance restrictions (e.g., intersection beyond a crest vertical curve), or engineering judgment indicates a significant conflict related to left-turning vehicles.</td></tr>
</table>

State	Material on Left-Turn Guidelines or Warrants
Nebraska http://www.dor.state.ne.us/roadway-design/pdfs/rwydesignman.pdf *Roadway Design Manual* July 2006	Left turn treatments may be necessary on two-lane highways where traffic volumes are high and safety considerations are sufficient to warrant them. Left turn lanes should be provided on divided arterials at intersections and at other median breaks where left turn volumes and/or vehicle speeds are high. The "Intersection Channelization Design Guide" (Transportation Research Board, "Intersection Channelization Design Guide," National Cooperative Highway Research Program Report 279, Washington, DC, 1994.) provides warrants and guidelines for auxiliary lane design.
Nevada http://www.nevadadot.com/reports_pubs/	**Reviewed main website page but did not identify link that would appear to have the left-turn warrants/guidelines material**

State	Material on Left-Turn Guidelines or Warrants
Nevada http://www.nevadadot.com/business/forms/pdfs/TrafEng_AccesMgtSysStandards.pdf *Access Management System and Standards* July 1999	**4.8 Left Turn Lane Requirements, Two Lane Unsignalized Roads** Table 4.8 [**table is a reproduction of the *Green Book* values for 40, 50 and 60 mph—Nevada's table also includes data for 70 mph**] lists the projected 20 year design-hour volumes and the operating speeds of traffic which necessitate the installation of left turn lanes. The traffic volumes to be considered in making this determination are the opposing (oncoming) traffic volumes, the advancing traffic volumes and the percent of advancing traffic which is turning left. Turn lanes may be required at lower volumes, by a traffic impact study or by the Department, to protect the traveling public. **4.9 Left Turn Lane Requirements, Four Lane, Undivided, Unsignalized Roads** Table 4.9 **[see below]** lists the projected 20 year design-hour volumes of traffic which necessitate the installation of left turn lanes on multilane, undivided, unsignalized roads. The traffic volumes which are to be considered in making this determination are the opposing (oncoming) traffic volumes, the advancing traffic volumes, and the percent of advancing traffic which is turning left. Turn lanes may be required at lower volumes, by a traffic study or by the Department, to protect the traveling public.

Table 4.9 Left-Turn Lane Requirements for Multilane Roads (Unsignalized)

Opposing Volume (ddhv)	Advancing Volume (ddhv) for left turn percentages of			
	5%	**10%**	**20%**	**30%**
800	140	110	80	70
600	220	160	120	100
400	350	250	190	160
200	530	380	290	250
100	650	480	350	310

4.10 Left Turn Lane Requirements, Four Lane, Divided, Unsignalized Roads

Table 4.10 **[see below]** lists the projected 20 year design-hour volumes of traffic which necessitate the installation of left turn lanes on divided, unsignalized, multilane roads. The traffic volumes which are to be considered in making this determination are the opposing (oncoming) traffic volumes, the advancing traffic volumes, and the percent of advancing traffic which is turning left. Turn lanes may be required at lower volumes, by a traffic study or by the Department, to protect the traveling public.

Table 4.10 Left-Turn Lane Requirements for Multilane Divided Roads (Unsignalized)

Opposing Volume (ddhv)	Advancing Volume (ddhv) for left turn percentages of			
	5%	**10%**	**20%**	**30%**
800	210	150	110	100
600	340	240	180	150
400	520	380	290	250
200	800	580	440	390
100	1000	720	550	480

State	Material on Left-Turn Guidelines or Warrants
New Hampshire http://www.nh.gov/dot/bureaus/highwaydesign/nhhighwaydesignmanuals.htm *Highway Design Manual* Updated URL: http://www.nh.gov/dot/org/projectdevelopment/highwaydesign/designmanual/index.htm	**Did not find discussion on left-turn lane warrants/guidelines**
New Jersey http://www.state.nj.us/transportation/eng/documents/RDME/sect6E2001.shtm *Roadway Design Manual*, Section 6: At-Grade Intersections December 27, 2002 Updated URL: http://www.state.nj.us/transportation/eng/documents/RDM/sec6.shtm *Roadway Design Manual*, Section 6: At-Grade Intersections March 2009	**6.06 Median Left-Turn Lane** **6.06.1 General** A median lane is provided at an intersection as a deceleration and storage lane for vehicles turning left to leave the highway. Median lanes may be operated with traffic signal control, with stop signs, or without either, as traffic conditions warrant. Figure 6-T [not included] shows a typical median left-turn lane.

State	Material on Left-Turn Guidelines or Warrants
New Mexico http://nmshtd.state.nm.us/main.asp?secid=11703 *State Access Management Manual* September 2001	(see tables below)

Table 17.B-1. Criteria for Deceleration Lanes on URBAN TWO-LANE HIGHWAYS

Turning Volume[1] (veh/hr)	LEFT-TURN DECELERATION LANE Minimum Directional Volume in the Through Lane (veh/hr/ln)[2] ≤ 30 mph	35-45 mph	45-55 mph
< 5	Not Required	Not Required	Not Required
5	510	450	330
10	390	330	210
15	320	250	150
20	270	200	120
25	230	160	100
30	200	130	Required
35	170	110	Required
40	150	Required	Required
45	130	Required	Required
≥ 46	Required	Required	Required

Left-turn Deceleration Lanes are Required on Urban Two-Lane Highways for the following Left-turn Volumes:

- ≤ 30 mph : 46 veh/hr or more
- 35 to 40 mph : 36 veh/hr or more
- 45 to 55 mph : 26 veh/hr or more

Notes:
1. *Use linear interpolation for turning volumes between 5 and 45 veh/hr.*
2. *The directional volume in the through lane includes through vehicles and turning vehicles.*

Table 17.B-2 Criteria for Deceleration Lanes on URBAN MULTI-LANE HIGHWAYS

Turning Volume[1] (veh/hr)	LEFT-TURN DECELERATION LANE Minimum Directional Volume in the Adjacent Through Lane (veh/hr/ln)[2] ≤ 30 mph	35-45 mph	45-55 mph
< 5	Not Required	Not Required	Not Required
5	Not Required	490	420
10	420	370	300
15	360	290	220
20	310	230	160
25	270	190	130
30	240	160	110
35	210	130	100
40	180	120	Required
45	160	110	Required
≥ 46	140	Required	Required

Left-turn Deceleration Lanes are Required on Urban Multi-Lane Highways for the following Left-turn Volumes:

- ≤ 30 mph : 56 veh/hr or more
- 35 to 40 mph : 46 veh/hr or more
- 45 to 55 mph : 36 veh/hr or more

Notes:
1. *Use linear interpolation for turning volumes between 5 and 55 veh/hr.*
2. *The directional volume in the adjacent through lane includes through vehicles and turning vehicles.*

State	Material on Left-Turn Guidelines or Warrants
New Mexico (continued)	(Tables 17.B-3 and 17.B-4 below)

Table 17.B-3
Criteria for Left-turn Deceleration Lanes on
RURAL TWO-LANE HIGHWAYS

Left-Turn Volume[1] (veh/hr)	LEFT-TURN DECELERATION LANE			
	Minimum Directional Volume in Through Lane (veh/hr/ln)[2]			
	≤ 30 mph	35-40 mph	45-55 mph	> 55 mph
< 5	Not Required	Not Required	Not Required	Not Required
5	400	20	120	60
10	240	140	80	40
15	160	100	60	Required
20	120	80	Required	Required
25	100	Required	Required	Required
≥ 26	Required	Required	Required	Required

Left-turn Deceleration Lanes are Required on Rural Two-Lane Highways for the following Left-turn Volumes:

- ≤ 30 mph : 26 veh/hr or more
- 35 to 40 mph : 21 veh/hr or more
- 45 to 55 mph : 16 veh/hr or more
- > 55 mph : 11 veh/hr or more

Notes:
1. *Use Linear interpolation of left-turn volumes between 5 and 25 veh/hr.*
2. *The directional volume in the through lane includes through vehicles and turning vehicles.*

Table 17.B-4
Criteria for Left-turn Deceleration Lanes on
RURAL MULTI-LANE HIGHWAYS

Left-Turn Volume[1] (veh/hr)	LEFT-TURN DECELERATION LANE			
	Minimum Directional Volume in Through Lane (veh/hr/ln)[2]			
	≤ 30 mph	35-40 mph	45-55 mph	> 55 mph
< 5	Not Required	Not Required	Not Required	Not Required
5	400	20	120	60
10	240	140	80	40
15	160	100	60	Required
20	120	80	Required	Required
25	100	Required	Required	Required
30	130	Required	Required	Required
35	110	Required	Required	Required
≥ 36	Required	Required	Required	Required

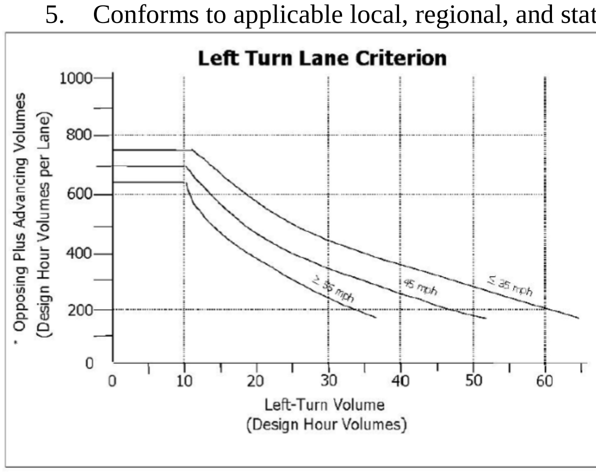
Left-turn Deceleration Lanes are Required on Rural Multi-Lane Highways for the following Left-turn Volumes:

- ≤ 30 mph : 36 veh/hr or more
- 35 to 40 mph : 26 veh/hr or more
- 45 to 55 mph : 21 veh/hr or more
- > 55 mph : 16 veh/hr or more

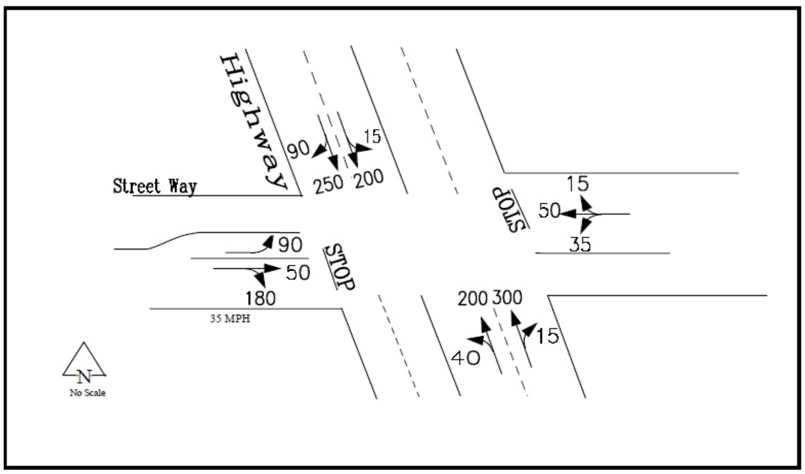
Notes:
1. *Use Linear interpolation of left-turn volumes between 5 and 35 veh/hr.*
2. *The directional volume in the through lane includes through vehicles and turning vehicles.*

State	Material on Left-Turn Guidelines or Warrants
New York https://www.nysdot.gov/divisions/engineering/design/dqab/hdm/hdm-repository/chapt_05.pdf *Highway Design Manual*, Chapter 5—Basic Design August 23, 2006	The decision to construct left-turn lanes should consider: • The volume of left-turning traffic and the volume of opposing traffic. In some cases, capacity analysis may clearly indicate a need for left-turn lanes. Exhibit 9-75 in Chapter 9 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2004, includes traffic volume criteria to be considered in determining the need for left turn lanes along two-lane highways. • The accident history. An accident pattern of rear-end accidents involving queued left turners or vehicles turning left in front of opposing traffic is often mitigated by exclusive left-turn lanes. NYSDOT accident reduction factors show an average reduction of around 30% when a left-turn lane is installed and is an appropriate alternative to mitigate a left-turn accident problem. • The accident potential and the anticipated operating speeds (i.e., the possible severity of an accident). • Sight distance on the mainline affecting the ability to see a vehicle waiting to turn. • The construction costs. • The right of way impacts.
North Carolina http://www.ncdot.org/doh/preconstruct/altern/value/manuals/RDM2001/part1/chapter9/pt1ch9.pdf *Roadway Design Manual*, Chapter 9: At Grade Intersections	**Found discussion on right-turn lane warrants but not left-turn lane warrants**
North Dakota *Design Manual* http://www.dot.nd.gov/manuals/design/designmanual/designmanual.htm	**Reviewed main website page but did not identify link that would appear to have the left-turn warrants/guidelines material**

State	Material on Left-Turn Guidelines or Warrants
Ohio http://www.dot.state.oh.us/Divisions/ProdMgt/Roadway/roadwaystandards/Location%20and%20Design%20Manual/400_jul06.pdf *Location and Design Manual* Section 400 Intersection Design July 2006 Updated URL: *Location and Design Manual, Volume 1 Roadway Design* (October 2010) http://www.dot.state.oh.us/Divisions/ProdMgt/Roadway/roadwaystandards/Pages/locationanddesignmanuals.aspx	**401.6 Approach Lanes** **401.6.1 Left Turn Lanes** Probably the single item having the most influence on intersection operation is the treatment of left turn vehicles. Left turn lanes are generally desirable at most intersections. However, cost and space requirements do not permit their inclusion in all situations. Intersection capacity analysis procedures of the current edition of the Highway Capacity Manual should be used to determine the number and use of left turn lanes. For unsignalized intersections, left turn lanes may also be needed if they meet warrants as provided in Figures 401-5a, b, and c. The warrants apply only to the free-flow approach of the unsignalized intersection. **[Some of the curves in Figures 401-5a and b are based on *Green Book* values. The graphs include several additional percent left turn values than included in the *Green Book*. Figure 401-5c is for four-lane highways with curves for divided and undivided roadways. The source for curves is not apparent.]**
Oklahoma *Roadway Design Standards and Specifications* http://www.okladot.state.ok.us/roadway/standards.htm	**Did not find any discussion on left-turn warrants**
Oregon ftp://ftp.odot.state.or.us/techserv/roadway/web_drawings/HDM/Rev_E_2003Chp09.pdf http://egov.oregon.gov/ODOT/HWY/ENGSERVICES/hwy_manuals.shtml#2003_English_Manual *Highway Design Manual*, 2003 English Manual, Chapter 9 (Intersection and Interchange Design)	**Left Turn Lanes** Providing a left turn lane at an intersection will significantly improve the safety of the intersections. Eliminating conflicts between left turning vehicles decelerating or stopping and through traffic is an important safety consideration. A left turn lane must be provided at all non-traversable median openings. Left turn lanes may be installed at intersections meeting the installation criteria. The left turn lane installation criteria are different for signalized and unsignalized intersections. Refer to Section 9.3 Signalized Intersections, and Section 9.4 Unsignalized Intersections, for the appropriate siting criteria. **9.4 Unsignalized Intersections, Left Turn Lanes** Left turn lanes at unsignalized intersections must meet the siting criteria to justify installation. Regardless of the funding source, the Region Traffic Engineer must approve all unsignalized channelized left turn lanes. The design should work with the Traffic Management Section in locations where left turn lanes are being considered. Left turn siting criteria has been established and is located in Appendix F along with a left and right turn lane siting example.

State	Material on Left-Turn Guidelines or Warrants
Oregon (continued)	**Appendix F — Left and Right Turn Lane Siting Criteria, Left Turn Lane Criteria** Purpose A left turn lane improves safety and increases the capacity of the roadway by reducing the speed differential between the through and the left turn vehicles. Furthermore, the left turn lane provides the turning vehicle with a potential waiting area until acceptable gaps in the opposing traffic allow them to complete the turn. Installation of a left turn lane must be consistent with the access management strategy for the roadway. Left Turn Lane Evaluation Process 1. A left turn lane should be installed if criteria 1 (Volume), or 2 (Crash), or 3 (Special Case) are met, unless a subsequent evaluation eliminates it as an option, and 2. The Region Traffic Engineer must approve all proposed left turn lanes on state highways, regardless of funding source, and 3. The State Traffic Engineer shall review and approve all proposed left turn lanes at signalized intersection locations on State Highway System to ensure proper signal operation, prior to design and construction, and 4. Complies with Access Management Spacing Standards, and 5. Conforms to applicable local, regional, and state plans.  * ((Advancing volume/number of advancing through lanes) + (opposing volume/ number of opposing through lanes)) **I Criterion 1: Vehicular volume** The vehicular volume criterion is intended for application where the volume of intersecting traffic is the principal reason for considering installation of a left turn lane. The volume criteria is determined by the Texas Transportation Institute (TTI) curves in Figure F-1 **[The graph is similar to work done by Hawley and Stover, which is based on delay and conflict avoidance]**. The criterion is not met from zero to ten left turn vehicles per hour, but indicates that careful consideration be given to installing a left turn lane due to the increase potential for accidents in the through lanes. While the turn volumes are low, the adverse safety and operations impact may require installation of a left turn. The final determination will be based on a field study.

State	Material on Left-Turn Guidelines or Warrants
Oregon (continued)	**II Criterion II: Crash experience** The crash experience criterion is satisfied when: 1. Adequate trial of other remedies with satisfactory observance and enforcement has failed to reduce the accident frequency; and 2. A history of crashes of the type susceptible to correction by a left turn lane; and 3. The safety benefits outweigh the associated improvement costs; and 4. The installation of the left turn lane does not adversely impact the operations of the roadway. **III Criterion 3: Special Cases** 1. <u>Railroad crossings</u> — If a railroad is parallel to the roadway and adversely affects left turns, a worst case scenario should be used in determining the storage requirements for the left turn lane design. Other surrounding conditions, such as a drawbridge, could adversely affect left turns and must be treated in a similar manner. The left turn lane storage length depends on the amount of time the roadway is closed, the expected number of vehicle arrivals, and the location of the crossing or other obstruction. The analysis should consider all the variables influencing the design of the left turn lane, and may allow a design for conditions other than the worst case storage requirements, providing safety is not compromised. 2. <u>Passing lane</u> — Special consideration must be given to installing a left turn lane for those locations where left turns may occur and other mitigation options are not acceptable. 3. <u>Geometric/safety concerns</u> — Consider sight distance, alignment, operating speeds, nearby access movements, and other safety related concerns. 4. <u>Non-traversable median</u> — As required in the Median Policy, a left turn lane must be installed for any break in a non-traversable median. 5. <u>Signalized intersection</u> — Consideration shall be given to installing left turn lanes at signalized intersections. The State Traffic Engineer shall review and approve all proposed left turn lanes at signalized intersection locations on the state highway system. **IV Evaluation Guidelines** 1. The evaluations should indicate the installation of a left turn lane will improve the overall safety and/or operation of the intersection and the roadway. If these requirements are not met, the left turn lane should not be installed or, if already in place, not allowed to remain in operation. 2. Alternatives Considered — list all alternatives that were considered, including alternative locations. Briefly discuss alternatives to the left turn lane considered to diminish congestion/delays resulting in criteria being met. 3. Access management — address access management issues such as the long term access management strategy for the state roadway, spacing standards, other accesses that may be located nearby, breaks in barrier/curb, etc. 4. Land Use Concerns — Include how the proposed left turn lane address land use concerns and transportation plans. 5. Plan — Include a plan or diagram of proposed location of left turn lane 6. Operational requirements — consider storage length requirements, deceleration distance, desired alignment distance, etc. For signalized intersections, installing a left turn lane must be consistent with the requirements in the Traffic Signal Guidelines.

State	Material on Left-Turn Guidelines or Warrants
Oregon (continued)	*Volume Criterion Example* Figure B-7 shows an unsignalized intersection with a shared through-right lane and a shared through-left lane on the Highway. The peak-hour volumes and lane configurations are included in the figure. The 85th percentile speed is 45 mph and the intersection is located in a city with a population of 60,000. ***Southbound:*** The southbound advancing volume is 555 (90 + 250 + 200 + 15) and the northbound opposing volume is 515 vehicles (the opposing left turns are not counted as opposing volumes). The volume for the y-axis on Figure B-6 is determined using the equation: y-axis volume = (*Advancing Volume/Number of Advancing Lanes) + (Opposing Volume/ Number of Opposing Lanes)* y-axis = (555/2 + 515/2) = 535 To determine if the southbound left turn volume criteria is met, use the 45 mph curve in Figure B-6, 535 for the y-axis, and 15 left turns for the x-axis. The volume criterion is not met in the southbound direction. ***Northbound:*** The northbound advancing volume is 555 (40 + 200 + 300 + 15) and the southbound opposing volume is 540 vehicles (the opposing left turns are not counted as opposing volumes). The volume for the y-axis on Figure B-6 is (555/2+ 540/2) = 548. To determine if the northbound left turn volume criteria is met, use the 45 mph curve in Figure B-6, 548 for the y-axis, and 40 left turns for the x-axis. The volume criterion is met in the northbound direction.  **Figure B-7. Oregon, Sample Intersection for Volume Criterion Example.**

State	Material on Left-Turn Guidelines or Warrants
Pennsylvania ftp://ftp.dot.state.pa.us/public/Bureaus/design/Pub13M/Chapters/Chap03.pdf ftp://ftp.dot.state.pa.us/public/Bureaus/design/Pub13M/Chapters/Chap01.pdf *Design Manual,* Part 2 Highway Design, Chapter 3 Intersections, Chapter 1 General Design June 2007	**H. Direct and Indirect Left Turns and U-Turns.** The various design methods and arrangements to accommodate left-turn and U-turn movements are predicated on the design control dimensions (width of median and width of crossroad or street) and the size of vehicle used for design control. The necessity to turn left or to make a U-turn in the urban or heavily developed residential or commercial sectors represents serious problems with respect to safety and efficient operations. Although warrants for the use of speed-change lanes cannot be stated definitely, the general conclusions and considerations for their use are contained in *Design Manual*, Part 2, Chapter 1, Section 1.6. **1.6 ACCELERATION AND DECELERATION (SPEED-CHANGE) LANES** The term speed-change lane, acceleration lane or deceleration lane, as used herein, applies broadly to the added pavement joining the traveled way of the highway with that of the turning roadway and does not necessarily imply a definite lane of uniform width. The warrants for the use of speed-change lanes cannot be stated definitely. However, based on observations and past experience, the following general conclusions have been made: **1.** Speed-change lanes are warranted on high-speed and on high-volume highways where a change in speed is necessary for vehicles entering or leaving the through-traffic lanes. **2.** All drivers do not use speed-change lanes in the same manner. **3.** Use of speed-change lanes varies with volume, the majority of drivers using them at high volumes. **4.** The directional type of speed-change lane consisting of a long taper fits the behavior of most drivers and does not require maneuvering on a reverse-curve path. **5.** Deceleration lanes on the approaches to intersections that also function as storage lanes for turning traffic are particularly advantageous, and experience with them generally has been favorable. For additional information on speed-change lanes as applicable to intersections and interchanges, refer to the AASHTO *Green Book*, Chapter 9 and Chapter 10 and *Design Manual*, Part 2, Chapter 3 and Chapter 4.
Rhode Island https://www.pmp.dot.ri.gov/PMP/DesktopDefault.aspx?aM=udoc&oM=pages&c1P=info&c1I=833&appindex=0&appid=0&podid=-1&mth=1#pageAnchor6 *Highway Design Manual* 2008	**Left-turn lanes not mentioned other than discussion on lane widths**
South Carolina	**Reviewed main website page but did not identify link that would appear to have the left-turn warrants/guidelines material**

State	Material on Left-Turn Guidelines or Warrants
South Dakota http://www.sddot.com/pe/roaddesign/docs/rdmanual/rdmch12.pdf *Road Design Manual*, Chapter 12: Intersections 2007	**Left Turn Lane Criteria — Unsignalized Intersections** Left turn lanes should be provided where through and turning volumes create an operational or a potential accident problem. **Left Turn Lane Evaluation Process** • A left turn lane should be installed if Criteria 1 (Volume), or 2 (Crash), or 3 (Special Cases) are met, unless a subsequent evaluation eliminates it as an option; and • The left turn lane complies with access management spacing standards; and • The left turn lane conforms to applicable local, regional and/or state design guidelines. **[See the manual for specific criteria. South Dakota's graph references the Oregon DOT procedures manual.]**
Tennessee http://www.tdot.state.tn.us/Chief_Engineer/assistant_engineer_design/design/DGpdf/ENGLISH%20GUIDELINES.pdf TDOT *Roadway Design Guidelines* March 2006 Updated URL: *Roadway Design Guidelines* (August 2008) http://www.tdot.state.tn.us/Chief_Engineer/assistant_engineer_design/design/DGpdf/DESIGN%20GUIDELINE.pdf	To determine a warrant for and a required storage length, use the attached charts (Figures 2-13 through 2-16f) by M. D. Harmelink. (See also *A Policy of Geometric Design of Highways and Streets 2004*, Exhibit 9-75, page 685. This table is a condensed version of the Harmelink charts for two-lane highways.) The first chart applies to four-lane highways, all speeds [Figure 2-13]. The remaining charts are a function of speed and the percentage of lefts in the approaching traffic, and are applicable only to two-lane highways [Figures 2-14 to 2-16f].
Texas http://onlinemanuals.txdot.gov/txdotmanuals/rdw/rdw.pdf *Roadway Design Manual* October 2006	Left-turn lanes on two-lane highways at intersecting crossroads generally are not economically justified. For certain moderate or high volume two-lane highways with heavy left-turn movements, however, left-turn lanes may be justified in view of reduced road user accident costs. Figure 3-11 provides recommendations for when left-turn lanes should be considered based on traffic volumes**. [Includes table with values similar to *Green Book* values.]**

State	Material on Left-Turn Guidelines or Warrants
Utah http://www.udot.utah.gov/main/f?p=100:pg:4147010608516685::::V,T:,1498 *Roadway Design Manual of Instruction* May 2007	Highway Capacity Provide a level of service C or higher for a 20-year design in a rural area and a level of service D or higher for a 20-year design in an urban area. Decisions from going through the environmental process and implementing CSS may impact the decision of which level of service to provide. Design all elements of the roadway (including intersections) to the selected level of service. **The need for channelization, left-turn lanes, etc., is directly related to the acceptable level of service.** The *Highway Capacity Manual* presents a more thorough discussion of the level-of-service concept. It also supplies the analytical base for design calculations and decisions including capacity analysis. 7.18 Lane Types Auxiliary Lanes Under conditions of relatively high traffic volumes, traffic congestion problems can be significantly alleviated with auxiliary lanes to handle turning movements. In rural areas, consider left-turn lanes where there are 25 or more left-turn movements from the main highway in the peak hour.
Vermont http://www.aot.state.vt.us/progdev/standards/statabta.htm *Vermont State Design Standards* October 22, 1997	**Did not find information on left-turn lanes**
Virginia http://www.extranet.vdot.state.va.us/locdes/Electronic%20Pubs/2005%20RDM/appendc.pdf *Road Design Manual,* Appendix C: Section C-1—Design Features, Virginia Department of Transportation, Location and Design Division July 2008	**LEFT-TURN LANES** As a general policy, left-turn lanes are to be provided for traffic in both directions in the design of all median crossovers on non-access controlled four-lane or greater divided highways using controls as shown in Figure C-1-1 [not included] and adjusted upward as determined by Figure C-1-1.1 or by capacity analysis for left-turn storage. Left-turn lanes should also be established on two-lane undivided highways where needed for storage of left-turn vehicles and/or prevention of thru-traffic delay as shown in Figure C-1-1 and adjusted upward as determined by Table C-1-2 [not included] and Figure C-1-1.2 [not included] through C-1-1.19 [not included] or by capacity analysis for left-turn storage. See Table C-1-2.1 [not included] for TRUCK ADJUSTMENTS. In general, when left-turn volumes are higher than 100 veh/hr, an exclusive left-turn lane shall be considered. Figures C-1-1.2 through C-1-1.19 provide warrants for left-turn storage lanes on two-lane highways based on 5 to 30 percent left-turn volumes and operating speeds of 40, 50, and 60 MPH. Table C-1-2.1provides the additional storage length required for 10 to 50 percent truck volumes. These figures were derived from Highway Research Report No. 211. This study was undertaken to provide consistent volume warrants for left-turn storage lanes at unsignalized intersections. Intersections with poor visibility and/or a bad accident record may require the designer to use engineering judgment when volume conditions alone do not warrant a storage lane. **[COMMENTS: Warrants for left-turn storage lanes on two-lane highways table (Table C-1-2) is a reproduction of the *Green Book* table. HRR No. 211 = Harmelink's paper]**

State	Material on Left-Turn Guidelines or Warrants
Washington http://www.wsdot.wa.gov/Publications/Manuals/M22-01.htm *Design Manual* January 2006	***(2) Left-Turn Lanes and Turn Radii*** Left-turn lanes provide storage, separate from the through lanes, for left-turning vehicles waiting for a signal to change or for a gap in opposing traffic. (See 910.07(4) for a discussion on speed change lanes.) Design left-turn channelization to provide sufficient operational flexibility to function under peak loads and adverse conditions. (a) **One-Way Left-Turn Lanes** are separate storage lanes for vehicles turning left from one roadway onto another. When recommended, one-way left-turn lanes may be an economical way to lessen delays and accident potential involving left-turning vehicles. In addition, they can allow deceleration clear of the through traffic lanes. When evaluating left-turn lanes, include impacts to all intersection movements and users. At signalized intersections, use a traffic signal analysis to determine whether a left-turn lane is needed and what the storage requirements are (see Chapter 850). At unsignalized intersections, use the following as a guide to determine whether or not to provide one-way left-turn lanes: • A traffic analysis indicates that a left-turn lane will reduce congestion. On two-lane highways, use Figure 910-12a, based on total traffic volume (DHV) for both directions and percent left-turn traffic, to determine whether further investigation is needed. On four-lane highways, use Figure 910-12b to determine whether a left-turn lane is recommended. • An accident study indicates that a left-turn lane will reduce accidents. • Restrictive geometrics require left-turning vehicles to slow greatly below the speed of the through traffic. • There is less than decision sight distance at the approach to the intersection. An HCM analysis may also be used to determine whether left-turn lanes are necessary to maintain the desired level of service. Determine the storage length required on two-lane highways by using Figures 910-13a through 13c. On four-lane highways, use Figure 910-12b. These lengths do not consider trucks. Use Figure 910-7 for storage length when trucks are present. **[Figure 910-12a for two-lane unsignalized intersections appears to be a drawing using *Green Book* numbers. Figure 910-12b for four-lane unsignalized intersections may be from *NCHRP Report 279*/Harmelink.]**
West Virginia *Design Directives Manual* (October 2006) http://www.transportation.wv.gov/highways/engineering/DD/2006%20DD%20Manual%20MASTER.pdf	**Left-turn lanes not mentioned**
Wisconsin *Design Manual*	**Left-turn lanes not mentioned**
Wyoming *Road Design Manual*	**Left-turn lanes not mentioned**

APPENDIX D

INTERVIEW QUESTIONS

PLANNING

1. Do you provide left-turn treatments at unsignalized intersections? If so, please give some examples (e.g., bypass lanes, TWLTL, etc.).
2. Please provide details on your process for determining where left-turn treatments should be installed at unsignalized locations (e.g., who is involved, what affects the decision, etc.).
3. Does this process vary depending upon whether the left-turn treatment is needed for (a) a new development, (b) as part of a reconstruction project, or (c) as a spot improvement (e.g., based on crash experience)? If so, how?
4. When is inclusion of a left-turn accommodation considered on a project (e.g., during project scoping, during design, or after completion of a traffic study)?
5. What criteria do you use to determine whether a left-turn lane should be provided at an unsignalized location? (Please check all that apply.)
    ___ Harmelink’s guidelines
    ___ state DOT guidelines
    ___ local jurisdiction guidelines
    ___ none
    ___ other ___________________
6. Do the criteria vary if the site is in an urban or rural area? Or by speed limit? Or if the road is divided or undivided? If so, how?
7. If other left-turn accommodations (such as a bypass lane) are an option, what criteria do you use to determine where they should be provided?
8. If your local agency uses its own installation guidelines, how do they vary from state DOT guidelines? Would you be willing to provide us a copy?
9. Have your recent left-turn lane installations at unsignalized locations tended to be retrofit/restriping projects* or full-width build-out projects**? Why?
10. Have your recent left-turn lane installations at unsignalized locations tended to be for new developments, general street/highway improvements, or specific treatments to improve safety/reduce crashes? Why?
11. What methods and/or measures, if any, do you use to evaluate the effectiveness of unsignalized left-turn lanes or other accommodations (e.g., before-after study)?

DESIGN

12. To what extent do you encourage positive offset of your left-turn lanes?
13. How do you determine the appropriate values for design elements of a left-turn lane (e.g., queue length, storage length, taper length, lane width, offset, sight distance, etc.)?
14. How do you determine the appropriate values for design elements of other left-turn accommodations?

15. What is the cost of a typical left-turn lane installation for a retrofit? For a full-width build-out? For other left-turn accommodations?
16. Is there a cross section on which you would not consider installing a left-turn lane at an unsignalized location?

LEGAL/POLICY/FINANCE

17. Who pays for the installation of a left-turn treatment at (a) a driveway to a new development and (b) an intersection that is projected to be impacted by a new development (i.e., not at a driveway for the development)?
18. If construction of a left-turn lane would require obtaining property from a third party, how would that property be acquired and by whom?
19. For proposed developments, are the costs for left-turn lanes treated any differently than other mitigation needed to accommodate the additional site traffic?
20. Have there been any recent (i.e., within the last 5 years) changes in the decision-making process related to the installation of left-turn lanes or other accommodations at unsignalized intersections? Why? What were the changes?
21. Are you aware of the Supreme Court decisions relating to essential nexus or rough proportionality: *Nollan v. California Coastal Commission* and *Dolan v. City of Tigard*?
22. If yes, are you aware of any ramifications of these court cases as they relate to decisions on left-turn accommodations (including when to install and who pays)?

POTENTIAL FUTURE APPLICATIONS

23. What lessons have you learned that will guide future installations of left-turn accommodations at unsignalized intersections?
24. Are you considering making any changes to policies on this topic in your jurisdiction?
25. Are there any regulatory/policy changes needed or anticipated at other levels to deal with issues related to the installation of left-turn accommodations at unsignalized intersections?

* Could include restriping the roadway, changing parking regulations, or reducing or removing shoulders.
** Could include minor widening within the right-of-way or acquiring additional right-of-way.

APPENDIX E

INTERVIEWS

To help investigate the implementation of left-turn accommodations at unsignalized intersections, interviews were conducted of representatives from the following agencies/organizations:

- State DOTs,
- County governments,
- City governments, and
- Consultants.

The 25 questions in the interview were structured into the following four topics:

- Planning,
- Design,
- Legal/policy/finance, and
- Potential future applications.

The list of questions is included in Appendix D. This appendix presents the interview findings for each of the four topics listed above. Within the discussion of each topic, the results are discussed for the agencies/organizations included in the interviews (i.e., state DOT, county governments, city governments, and consultants).

The findings are based on interviews with 11 state DOT representatives, 5 counties, 4 cities, and 2 consultants. The 11 state DOT interview participants represented nine state DOTs; two of the state DOTs reflected in the survey had two interview participants—one state DOT had two different districts represented, and one state DOT had two different divisions within the central office represented.

PLANNING (QUESTIONS 1 to 11)

State DOT Practices

Process

All state DOT interview participants indicated that left-turn treatments are provided at unsignalized intersections. The treatments used by all state DOTs are left-turn lanes and two-way, left-turn lanes. Several participants noted that TWLTLs are typically used in areas where there may be poor access control. Four of the state DOTs indicated they consider bypass or shoulder widening but noted these techniques are typically used when there are no other viable options. Three of the state DOTs noted that they generally would not use bypass or shoulder widening. From the responses, there does not appear to be a clear distinction between the terms “left-turn bypass lanes” and “shoulder widening” (may be called “shoulder lanes”). Bypass lanes may be striped, but this does not appear to be universal. One state DOT noted that roundabouts are another treatment considered for dealing with left-turn movements.

The process for determining where left-turn treatments should be installed at unsignalized locations, who is involved, and what affects the decision may vary depending upon the circumstances involved and whether the question relates to:

- A developer seeking access onto the roadway system for a new development or major redevelopment,
- The state DOT reevaluating the roadway condition as part of an improvement project,
- A county- or city-initiated project that is being coordinated with the state DOT, or
- A problem location identified by citizen complaints or its crash rate.

One state DOT participant noted that the process must vary due to the limited timeframe the DOT staff has for reviewing driveway permit applications submitted by developers for access onto the state roadway system.

A developer is often required to submit a traffic impact study for a major proposed development. This study is intended to assess the roadway network's capability to accommodate the site-generated traffic. A major conclusion of the study is the mitigation to the roadways and intersections that is needed to handle the additional traffic. Left-turn treatments are often part of the identified mitigation. The DOT permitting staff or others involved in the study review play a critical role in the process to decide what treatment is needed. The design staff often leads the actual design effort that is done in accordance with the DOT's design manual.

For an improvement project, it typically would be the traffic engineering staff supporting the designers in helping to identify needed improvements based on existing or projected future conditions. Generally, the design staff plays a critical role in the process to decide what improvements should be made, including what left-turn treatments should be incorporated.

When a project is initiated by the local county or city, extensive coordination of the participating agencies is involved. The project could be related to the roadway system or economic development. The DOT staff involved would vary depending upon the nature and extent of the project.

When a problem location is identified, based on its poor safety or operating conditions, a team may be formed to investigate the causes and explore improvement options. The team considers the crash patterns, traffic volumes and movements, and physical conditions to help identify options for improving the traffic conditions.

All state DOTS responded to the question regarding when left-turn accommodations are considered in a highway reconstruction or improvement project. Seven state DOTs first consider left-turn accommodations during the planning or scoping of a project. Two state DOTs consider left-turn accommodations during the design phase of a project.

Criteria

Of the nine state DOTs represented in the interviews, three specified they use criteria based on the guidelines developed by Harmelink to determine whether a left-turn lane should be provided at an unsignalized location. A majority of the state DOTs referred to their own guidelines as the criteria used in the decision-making process. However, it appears that a number of these state DOT guidelines are derived from the research performed by Harmelink.

Based on responses from state DOTs that had more than one participant, the criteria may vary by district; one district replied that the Harmelink guidance was used, whereas the other district noted that state DOT guidelines were used. The criteria used by a state DOT may also vary based on the need being addressed. One state DOT had participants from both the traffic engineering unit and the driveway permitting unit. The traffic engineering unit would use, in the decision-making process related to left-turn accommodations, information on crash rates and patterns as well as on traffic volumes and conditions. The permitting unit would use state DOT guidelines regarding left-turn lane warrants.

The following are the responses to the question regarding whether the criteria for left-turn treatments vary if the site is in an urban or rural area, by speed limit, and/or whether it is located on a divided or undivided road:

- All of the above are taken into account.
- Speed limit does have an impact as well as whether the road is divided or not.
- Yes, the roadway design manual addresses all these varying conditions.
- The criteria for left-turn treatment may vary by the location setting.
- The criteria do not vary based on urban or rural area, speed limit, or divided/undivided road. However, typically there is a greater need to investigate these treatments in more rural or urbanizing areas.
- Criteria are essentially the same, but the design will vary.
- Since the treatment will vary on a case-by-case basis, all of the above need to be considered.
- Yes, they vary based on functional class and type of development. Also, speed for left-turn lanes on two-lane roads is considered.
- Urban/rural location is not a factor. Speed is not an issue with current criteria. The divided/undivided cross section needs to be considered.
- Urban/rural location is generally not a factor; however, urban areas are more difficult to deal with. High speed gets factored in qualitatively but is reflected in the deceleration length calculation. Divided/undivided road would have some effect.

The following are the responses to the question regarding what criteria are used for identifying when other left-turn accommodations (such as a bypass lane) should be provided:

- The respondent was not familiar with bypass lanes.
- The respondent's DOT does not really use bypass lanes.
- Requirements are set forth in the roadway design manual.
- The effectiveness of countermeasures, right-of-way requirements, cost, and schedule are considered.
- Respondent's DOT does not usually use other options.

- Respondent's DOT does not use bypass lanes. The DOT occasionally builds TWLTLs for low-speed conditions.
- Respondent's DOT would generally not use bypass lanes but may use TWLTLs.
- A bypass lane is not an option. The DOT has allowed a TWLTL in lieu of a left-turn lane where access is poorly controlled. The DOT has installed a median to block left-turn access.
- The roadway design manual has provisions for when bypass lanes should be installed and also has a discussion of TWLTLs.
- Bypass lanes are generally not used. TWLTLs are considered on a case-by-case basis.

Most state DOT participants did not know whether there are local agencies that use their own installation guidelines. One respondent indicated that the majority of counties and local governments use the state DOT guidance. One respondent indicated that one city does not use state DOT criteria; instead it uses criteria it established based mainly on a roadway's functional class.

Applications

According to a majority of the state DOTs represented in the interviews, recent left-turn lane installations at unsignalized locations have tended to include both retrofit/restriping projects and full-width build-out projects. The retrofit projects could involve restriping the roadway, changing parking regulations, or reducing or removing shoulders. The full-width projects could include minor widening within the right-of-way or acquiring additional right-of-way. One state DOT responded that its recent left-turn lane installations were retrofit projects. Three state DOTs indicated their recent left-turn lane installations were full-width projects. These three state DOTs offered more information regarding the full-width projects:

- Most are the addition of left-turn lanes at mostly rural county road intersections as part of the DOT's safety program. These were accomplished within the existing state right-of-way and involved minor widening of the pavement to accommodate the extra pavement width required to provide these turn lanes.
- The recent left-turn lane projects are on two-lane conventional roadways that require widening to accommodate the greater pavement width.
- In most cases full-width projects are done since retrofit has already been done where possible.

There was a range of responses to the question regarding whether recent left-turn lane installations at unsignalized locations tended to be for new developments, general street/highway improvements, or specific treatments to improve safety/reduce crashes.

The responses from state DOTs that had more than one participant varied. In one case, there were two districts responding, and in a second case there were two units from within the central office that were represented. For the state that had two districts responding, one district replied that left-turn lane installations have tended to be related to safety improvements, whereas the other district noted the installations appeared to be related to new developments. For the state that had two units responding, one unit replied that left-turn lane installations have tended to be related to new developments, highway improvements, and safety improvements, whereas the other unit noted the installations appeared to be related to new developments.

In response to the question regarding what methods and/or measures, if any, are used to evaluate the effectiveness of unsignalized left-turn lanes or other accommodations, the state DOTs noted that before and after studies or other evaluations are not generally performed. Safety improvements are the exception where evaluations are performed to identify the effectiveness of the improvement that was implemented.

County Practices

Process

All five county interview participants indicated that left-turn treatments are provided at unsignalized intersections. All counties surveyed use left-turn lanes. Three of the counties indicated they use TWLTLs. One noted that TWLTLs exist as a retrofit option when access management is not practical. Two counties indicated they consider bypass or shoulder widening but noted these techniques are typically used when options are limited.

The following are the responses to the question regarding the process used for determining where left-turn treatments should be installed at unsignalized locations, who is involved, and what affects the decision:

- Access management guidelines are used for evaluating the needs for new developments as well as for identifying improvements to include in roadway improvement projects. The director of transportation who is also the county highway superintendent or a designee would make the decision.
- Concurrency rules similar to those in Florida are part of the process. A number of different units would need to be involved, including design, traffic engineering, and property development. If the process is applied to a new development, coordination with the developer and associated traffic engineer is required.
- The process has two main components—one related to a special fund for making improvements and a second relating to new developments desiring access. The default option is to add a left-turn lane in a county improvement project. Current policy requires a developer to build a left-turn lane at a site driveway.
- The two main elements of the process relate to development review and traffic studies. When the need for a left-turn lane is identified based on a development review, the developer is usually conditioned to provide a dedicated left-turn lane. This is typically determined as part of a traffic impact study conducted by a traffic engineer for the developer. The need for a left-turn treatment may also be identified based on safety studies, traffic operational studies, and citizen complaints. These studies are a collaborative effort involving the traffic and capital divisions within public works.
- The access proposal for a new development is reviewed by county staff for safety and consistency with access management guidelines.

The following are the responses to the question regarding whether the process used for determining where left-turn treatments should be installed at unsignalized locations varies for a new development, reconstruction project, and/or spot improvement:

- Generally no. The same guidelines are used, and the same methodology is applied. For spot improvements, however, crashes need to be addressed.
- It does. New developments result in right-of-way questions. How far do the decision makers want to go in pushing for a left-turn lane? There needs to be more of a give and take. For a reconstruction project, however, it would likely be a county project, and the county would acquire the right-of-way needed for the desired level of improvement.
- Except for low-volume developments, a left-turn lane would be installed. Generally, the county would provide left-turn lanes or install a median to eliminate the left-turn movement. TWLTLs are not installed, and there are efforts to remove them and replace them with medians.
- If a road is being reconstructed, the county will add medians or left-turn lanes consistent with what would be required from a private developer.

All county participants responded to the question on when left-turn accommodations are considered in a highway reconstruction or improvement project. Four of the counties consider left-turn accommodations during the planning or scoping of a project. One county considers left-turn accommodations during the preliminary design phase of a project.

Criteria

The following are the responses to the question regarding the criteria used to determine whether a left-turn lane should be provided at an unsignalized location:

- The county uses a combination of Harmelink, state DOT guidelines, and local jurisdiction guidelines.
- Criteria are based on design considerations.
- The county uses Harmelink guidance and crash data.
- The county uses local jurisdiction guidelines.
- The county uses Harmelink guidance and local access management guidelines based on Harmelink.

The following are the responses to the question regarding whether the criteria for left-turn treatments vary if the site is in an urban or rural area, by speed limit, and/or whether it is located on a divided or undivided road:

- Speed limit is considered. The county is more likely to require a left-turn lane on a higher-speed road. Since the county does not have many divided roadways, this is not an issue.
- The criteria do not vary by speed limit, at least for minor approaches, and there is no difference for urban/rural areas. Since the county does not have many divided roadways, this is not an issue.
- The default decision is to install a left-turn lane. Higher speed is better justification for a left-turn lane. If the roadway is divided, there would not be a median opening unless the developer is able to justify it.
- Yes, the criteria are based on the Harmelink guidance

- No. However, if a proposed access or street is in a built-up area where roadway expansion is not practical or consistent with the corridor plan, then turn lanes may not be required.

The following are the responses to the question regarding what criteria are used for identifying when other left-turn accommodations (such as a bypass lane) should be provided:

- The county uses a combination of Harmelink and the access management guidelines with consideration of available gaps in the traffic stream.
- The county has discussed bypass lanes but has not used them.
- A bypass lane would be used only for maintenance.
- A bypass lane is not considered an option.
- Bypass lanes are only considered at T-intersections where turning volumes are not expected to cause queuing of multiple vehicles at any given time.

Two of the counties use state DOT installation guidelines for left-turn treatments. One county has established its own set of guidelines.

Applications

According to a majority of the counties represented in the interviews, recent left-turn lane installations at unsignalized locations have tended to include both retrofit/restriping projects and full-width build-out projects. The retrofit projects could involve restriping the roadway, changing parking regulations, or reducing or removing shoulders. The full-width projects could include minor widening within the right-of-way or acquiring additional right-of-way. Two counties indicated their recent left-turn lane installations were full-width projects.

All five county participants replied to the question regarding whether recent left-turn lane installations at unsignalized locations tended to be for new developments, general street/highway improvements, or specific treatments to improve safety/reduce crashes. Four counties replied that left-turn lane installations have tended to be related to a combination of new developments, highway improvements, and safety improvements. One county responded that the majority of installations appeared to be related to new developments.

In response to the question regarding what methods and/or measures, if any, are used to evaluate the effectiveness of unsignalized left-turn lanes or other accommodations, the counties noted that before and after studies or other evaluations are not generally performed. One county indicated they do evaluations of improvements done as part of safety projects. One county noted they sometimes do operational analysis or field assessment after an improvement is completed.

City Practices

Process

All four city interview participants indicated that left-turn treatments are provided at unsignalized intersections. All cities use left-turn lanes and TWLTLs. None of the cities indicated they use left-turn bypass lanes or shoulder widening.

The following are the responses to the question regarding the process used for determining where left-turn treatments should be installed at unsignalized locations, who is involved, and what affects the decision:

- A lot of the decisions are based on traffic studies for new developments.
- The traffic engineering staff is involved and make a decision based on left-turn volume, crashes, delay, and funding availability.
- Due to limited funding, there are not a lot of projects. When funds are available for major projects, roads are typically built to a three-lane or five-lane cross section.
- Typically, on divided roadways, left-turn treatments at unsignalized intersections are instituted in the design process by the roadway designer. On undivided roadways, left-turn treatments are typically added by the traffic engineer in response to increased left-turn traffic volumes and/or a high number of accidents.

The following are the responses to the question regarding whether the process used for determining where left-turn treatments should be installed at unsignalized locations varies for a new development, reconstruction project, and/or spot improvement:

- Yes. The process is essentially the same for new developments or reconstruction projects and is based primarily on operational analysis. For locations with higher crash rates, safety is the predominant focus.
- No.
- The decision to install a left-turn lane or TWLTL is based on volumes and crashes. Therefore, the process would be the same, although the funding would be an issue.
- Yes, left-turn pockets are required for all new developments generating more than 250 vehicle trips per day on an existing divided roadway. For reconstruction projects, left-turn pockets are provided at all unsignalized intersections on divided roadways; on undivided roadways, dedicated left-turn lanes or TWLTLs are rarely provided at unsignalized intersections. For spot improvements on undivided roadways, left-turn treatments are typically added by the traffic engineer in response to increased left-turn traffic volumes and/or a high number of accidents.

All city participants responded to the question regarding when left-turn accommodations are considered in a highway reconstruction or improvement project. Two participants stated that it was after a traffic study, one participant indicated during project scoping, and one participant indicated during the design process.

Criteria

The following are the responses to the question regarding the criteria used to determine whether a left-turn lane should be provided at an unsignalized location:

- The city uses a combination of state DOT guidelines and local jurisdiction guidelines.
- The city uses a combination of local jurisdiction guidelines, traffic impact studies, and staff site observations.
- The city uses local guidelines that reflect the street’s average daily traffic and operating speed, the driveway’s volume, and the driveway’s left-turn ingress volume as a percentage of the street’s peak-period traffic volume.
- The city uses local jurisdiction guidelines.

The following are the responses to the question regarding whether the criteria for left-turn treatments vary if the site is in an urban or rural area, by speed limit, and/or whether it is located on a divided or undivided road:

- Each situation is considered individually.
- No.
- Operating speed is considered.
- Typically, the criteria do not vary if the site is an urban versus rural area or by speed limit. However, they do vary based on whether the road is divided or undivided.

The following are the responses to the question regarding what criteria are used for identifying when other left-turn accommodations (such as a bypass lane) should be provided:

- Bypass lanes are not used.
- Bypass lanes are used by the state DOT in rural areas. They are typically not used in a city.
- Other left-turn treatments are not used.

One of the cities uses state DOT installation guidelines for left-turn treatments since most of its routes are state routes. One city has no formal written guidelines.

Applications

Two of the cities indicated their recent left-turn lane installations at unsignalized locations were predominantly full-width projects. The full-width projects could include minor widening within the right-of-way or acquiring additional right-of-way. One city indicated that recent left-turn lane installations have tended to include both retrofit/restriping projects and full-width build-out projects. The retrofit projects could involve restriping the roadway, changing parking regulations, or reducing or removing shoulders. One city indicated that their recent installations at unsignalized intersections tended to be restriping projects because of costs.

All four city participants replied to the question regarding whether recent left-turn lane installations at unsignalized locations tended to be for new developments, general street/highway improvements, or specific treatments to improve safety/reduce crashes. One city replied that left-turn lane installations have tended to be related to a combination of new developments, highway improvements, and safety improvements. One city responded that the majority of installations appeared to be related to new developments. One city indicated that the installations have been predominantly general improvement projects or safety projects. One city indicated their recent left-turn installations have been in response to increased left-turn traffic volumes and/or a high number of accidents.

In response to the question regarding what methods and/or measures, if any, are used to evaluate the effectiveness of unsignalized left-turn lanes or other accommodations, two of the cities noted they do before and after studies; one does studies related to crashes and delay, and one does benefit/cost analysis.

Consultant Practices

A consultant must follow the process and apply the criteria used by the jurisdiction for which (i.e., highway improvement) or in which (i.e., new development) a project is being done. Therefore, the responses from the two consultants to these questions of the survey are not included.

DESIGN (QUESTIONS 12 TO 16)

State DOT Practices

All state DOT representatives indicated that they have guidelines that they use for determining the appropriate values for design elements of a left-turn lane (e.g., queue length, storage length, taper length, lane width, offset, sight distance, etc.). These guidelines could be contained within a range of documents including a road design manual, highway design manual, access management guide, project development design manual, corridor management policy, manual on uniform traffic control devices, or other design standards.

Some of the state DOT representatives indicated they used state criteria for certain design elements and AASHTO guidance for others.

State DOT representatives generally cited the same guidelines for design elements of other left-turn accommodations as they did for left-turn lanes. The representatives from two of the state DOTs—Kansas and Minnesota—cited guidelines that were developed for the design of left-turn bypass lanes. (Although left-turn lanes are allowed by the Kansas DOT, they are not commonly used.) In its *Corridor Management Policy* (Section C: Access Criteria), the Kansas DOT has guidelines based on posted highway speed for left-turn bypass lanes at T-intersections. The Minnesota DOT, in its *Road Design Manual* (Section 5-4.0: Rural Intersections), has two guidelines for the dimensions of left-turn bypass lanes at T-intersections and 4-leg intersections in rural areas. The Minnesota DOT in the *Road Design Manual* indicates that left-turn lanes are "the most effective and safe way to separate the left-turning from the through traffic streams"; left-turn bypass lanes should be considered only when there is no other alternative left-turn treatment.

For the question regarding whether there is a cross section on which installing a left-turn lane at an unsignalized location would not be considered, several interview participants indicated there was no cross section on which they would not consider the installation of a left-turn lane. Other responses included:

- The *Road Design Manual* needed to be checked for all design-related questions.
- Installing the left-turn lane was more a function of site conditions than the cross section.
- There was no cross section on which installing a left-turn lane would not be considered unless the location was proven to be economically or technically infeasible.
- The DOT is reluctant to impact existing private development buildings.
- Installing the left-turn lane is more of a design question. A left-turn lane as narrow as 10 ft in an urban, low-speed area may be acceptable.

- Yes, but it depends on truck volumes. The DOT may accept a 10-ft left-turn lane on low-speed roadways.
- Yes, the DOT would not accept less than a 10-ft left-turn lane on a low-speed roadway and a 12-ft left-turn lane on a high-speed roadway.
- A 12-ft-wide left-turn lane is preferred. No less than 11 ft on a rural facility is acceptable. There was a situation in an urban area where the left-turn lane was 11 ft wide and the through lane was 10 ft wide.

About half of the interview participants did not know the cost of constructing left-turn treatments. They explained that where left-turn treatments are part of more comprehensive improvements, the cost of left-turn treatments is not computed separately. They further explained they would not be aware of the cost if a developer paid for the installation. The cost of constructing left-turn lanes on the major approaches at a four-leg intersection could range from $250,000 to $400,000 in urban areas with a shorter deceleration length. The cost in rural areas can be greater, ranging from $600,000 to $800,000, due to the higher speeds and greater length of the design elements. One respondent indicated the full-width build-out can be about $1,000,000 per intersection, including construction and right-of-way cost.

Based on the responses from the interview participants, there appears to be limited usage of positive offset left-turn lanes. Reasons for the limited usage include cost and a lack of familiarity. Only one respondent indicated that the state DOT always tries to achieve this design.

County Practices

All county interview participants indicated that they have guidelines that they use for determining the appropriate values for design elements of a left-turn lane (e.g., queue length, storage length, taper length, lane width, offset, sight distance, etc.). These guidelines could be contained within a range of formal and informal documents; two use their state DOT design manuals, one uses internal guidelines that have been established, one uses the Harmelink methodology and available right-of-way, and one uses a combination of internal guidelines and other references.

Three of the five county interview participants cited the same guidelines for design elements of other left-turn accommodations as they did for left-turn lanes—a state DOT design manual, internal guidelines, and a combination of internal guidelines and other references. Of the other two participants, one indicated this is not relevant, and one did not know how to reply.

The following are the responses to the question regarding whether there is a cross section on which installing a left-turn lane at an unsignalized location would not be considered:

- A left-turn lane would not be considered if there is inadequate width.
- A left-tune lane would not be considered if there is insufficient sight distance related to horizontal or vertical curves.
- The turn lane must be compatible with other geometry at the intersection. For example, an opposing left-turn lane should be present if the intersection has four legs.

Two of the interview participants provided an estimated cost of constructing left-turn treatments. The first indicated that his rule of thumb is between $75,000 and $100,000 for a left-turn lane at a T-intersection, $175,000 for two approaches at a four-leg intersection, and $300,000 for all four approaches. The second participant indicated that an estimated cost of several hundred thousand dollars is typical for left-turn lanes and that right-turn/bypass lanes may run about half the cost. Other participants explained that costs vary widely and that, where left-turn treatments are part of more comprehensive improvements, the cost of left-turn treatments is not computed separately. They further explained they would not be aware of the cost if a developer paid for the installation.

Based on the responses from the interview participants, there appears to be limited usage of positive offset left-turn lanes. Emphasis is placed on aligning the opposing left-turn lanes.

City Practices

Three of the four interview participants replied to this question regarding the guidelines they use for determining the appropriate values for design elements of a left-turn lane (e.g., queue length, storage length, taper length, lane width, offset, sight distance, etc.). One indicated AASHTO guidance, one identified an internal document that is used along with field observations, and one indicated that these design elements are specified in the city's pavement design manual. None of the respondents offered any insights into the guidelines used for design elements of other left-turn accommodations.

The following are the responses to the question regarding whether there is a cross section on which installing a left-turn lane at an unsignalized location would not be considered:

- The conditions at each intersection need to be considered, including sight distance, speed limit, other access points, etc.
- An 11-ft width would be preferable.
- No.

One of the consultants had the following response to the question regarding whether there is a cross section on which installing a left-turn lane at an unsignalized location would not be considered: in a central business district where the distance between curbs is 30 ft, the sidewalk width could not be reduced, buildings would not allow the pavement to be widened, and additional right-of-way could not be acquired.

One of the interview participants provided an estimated cost of constructing left-turn treatments of $300,000 to $1,000,000 for full intersection widening. Another participant indicated that the estimated cost for widening a two-block segment to include TWLTLs is $1,000,000. Another city indicated costs of $2,000 (retrofit [restriping]), $25,000 (full-width build-out [no existing median opening]), and $17,000 (full-width build-out [with existing median opening]).

Regarding the usage of positive offset left-turn lanes, one participant indicated the usage of positive offset is strongly encouraged, one replied that it is done, and two indicated it is not used.

Consultant Practices

The two consultant interview participants replied to this question regarding the guidelines they use for determining the appropriate values for design elements of a left-turn lane (e.g., queue length, storage length, taper length, lane width, offset, sight distance, etc.). One indicated the dimensions used without citing a reference. The second indicated that the values of the design elements are based on first considering sight distance and queue length. Regarding the guidelines used for design elements of other left-turn accommodations, one participant indicated the application of state standards based on speed. The second noted that the values of these design elements depend on the constraints involved that limit widening opportunities.

One of the consultant participants indicated that the usage of positive offset left-turn lanes is generally limited due to right-of-way constraints.

LEGAL/POLICY/FINANCE (QUESTIONS 17 to 22)

State DOT Practices

Who Pays for Left-Turn Treatment

Of the nine state DOTs represented in the interviews, six indicated unequivocally that the developer of a new development would be responsible for paying for the installation of a left-turn treatment at a **site driveway**. Two indicated that a developer would “normally” or “typically” be responsible. However, the Minnesota DOT cannot require a developer to pay for the installation of a left-turn lane at a site driveway. The following provisions in its *Access Management Manual* (Chapter 4, page 16) would apply:

> For private driveway permits: Conditions of approval must fall within the reasonable exercise of the state’s police power. Conditions related to restricted movements and turn lanes will include only the design of the driveway entrance itself to restrict movements (e.g., pork chop design) and construction of warranted right-turn lanes provided there is adequate existing right-of-way to accommodate the right-turn lane. The applicant will be responsible for the costs associated with constructing these entrance design features.
>
> Other design features related to medians or turn lanes, warranted under the guidelines, will not be treated as conditions of the driveway permit, but will be recommended for consideration by the Local Governmental Unit as locally-initiated improvements to the trunk highway system. The Local Governmental Unit will need to determine whether it can assign any of the costs of these improvements, including the acquisition or dedication of additional right-of-way, to private property owners through assessments or related zoning and subdivision approvals.

For an off-site improvement, such as an intersection that is projected to be impacted by a new development (but not a site driveway), there was a range of responses of who would be responsible for paying for the installation of a left-turn treatment. Almost all of the state DOTs indicated that the developer would be expected to have some level of participation. Several indicated that the developer would be responsible for the mitigation. Some indicated that the developer would be responsible to pay a “fair share” or “proportional” cost. Several indicated that the local government agency (possibly city or county) would be expected to participate in the funding. One indicated that the developer would pay for the full cost of the treatment if the need for the off-site improvement on opening day was driven by the development.

There was a wide range of responses to the question regarding who would obtain the property from a third party for the construction of a left-turn treatment. Several indicated the developer would need to take the lead in the coordination for obtaining the property. Two state DOT representatives indicated that it may be done in a number of ways that would vary depending upon the nature of the development and the improvement. Although most indicated that the DOT would leave it to other entities to coordinate this, one DOT responded that the agency could get involved. Another DOT indicated it would be up to the city in which the project is located. One DOT indicated it may take the lead in a rural area but would look for a city to take the lead in an urban area.

All state DOT representatives that were interviewed indicated that the costs for left-turn accommodations are not treated differently than other mitigation needed to handle projected site-generated traffic.

Policy

Interview participants from three of the nine state DOTs indicated there had been changes in the past 5 years related to the decision-making process for the installation of left-turn lanes or other accommodations at unsignalized intersections. One indicated that the DOT has shifted away from using the Harmelink graphs, for establishing the need for a left-turn lane, to more general guidelines that include safety.

Legal

Six of the eleven state DOT interview participants were not aware of the two Supreme Court decisions relating to essential nexus or rough proportionality—*Nollan v. California Coastal Commission* and *Dolan v. City of Tigard*. Five were aware of the Supreme Court decisions. Two of the state DOTs reflected in the survey had two interview participants—one state DOT had two different districts represented, and one state DOT had two different divisions within the central office represented. For both states with two representatives, one respondent was aware of the Supreme Court decisions, and one was not.

All of the interview participants who expressed an awareness of the Supreme Court decisions indicated they were aware of no ramifications of these court cases to state DOT decisions related to left-turn accommodations. Interview participants indicated that their policies are consistent with these decisions and already reflect rough proportionality, rational nexus, etc.

County Practices

Who Pays for Left-Turn Treatment

The interview participants from all five of the counties included in the interviews indicated the developer of a new development would be responsible for paying for the installation of a left-turn treatment at a site driveway.

For an off-site improvement, such as an intersection that is projected to be impacted by a new development (but not a site driveway), the following are the responses to the question regarding who would be responsible for paying for the installation of a left-turn treatment:

- The developer would be responsible.
- It would depend on the specific concurrency rules.
- It would be a political question.
- It would depend on if the impact was direct or indirect.
- It would vary: it could be the developer or a shared arrangement.

The following are the responses to the question regarding who would obtain the property from a third party for the construction of a left-turn treatment:

- The developer would need to take the lead in the coordination for obtaining the property for a large development.
- The county would acquire the property at the cost of the developer.
- The county may acquire the property if funds are available.
- The developer would acquire the property if the property is required for a left-turn lane needed for a development permit; otherwise, the county would do so.
- The county would acquire the property if it was a county project. The developer would acquire the property if the need was generated by a proposed development.

Four of the five county interview participants indicated that the costs for left-turn accommodations are not treated differently than other mitigation needed to handle projected site-generated traffic. One noted that there could be a difference related to whether the improvement was being made along the property frontage (of the development site) versus where other properties are involved.

Policy

All five of the county interview participants indicated there were no major changes made in the past 5 years related to the decision-making process for the installation of left-turn lanes or other accommodations at unsignalized intersections.

Legal

Two of the five county interview participants were aware of the two Supreme Court decisions relating to essential nexus or rough proportionality—*Nollan v. California Coastal Commission* and *Dolan v. City of Tigard*. Three were not aware of the Supreme Court decisions.

Both of the interview participants who expressed an awareness of the Supreme Court decisions indicated they were aware of no ramifications of these court cases to county decisions related to left-turn accommodations.

City Practices

Who Pays for Left-Turn Treatment

The interview participants from all four of the cities included in the interviews indicated the developer of a new development would be responsible for paying for the installation of a left-turn treatment at a site driveway.

For an off-site improvement, such as an intersection that is projected to be impacted by a new development (but not a site driveway), the following are the responses to the question regarding who would be responsible for paying for the installation of a left-turn treatment:

- The developer pays (indicated by two participants).
- The developer pays if the treatment is required by ordinance.
- The city pays, with some developer contribution.

The following are the responses to the question regarding who would obtain the property from a third party for the construction of a left-turn treatment:

- The developer is responsible.
- The city is responsible, either by way of dedication or purchase.
- It would depend. The developer or city may be responsible, depending upon whether the work would be considered a developer job or a city job.
- The developer must purchase the property and dedicate it to the city.

All four of the city interview participants indicated that the costs for left-turn accommodations are not treated differently than other mitigation needed to handle projected site-generated traffic.

Policy

Three of the four city interview participants indicated there were no major changes made in the past 5 years related to the decision-making process for the installation of left-turn lanes or other accommodations at unsignalized intersections. One noted that there are more innovative left-turn treatments as a result of successful applications elsewhere.

Legal

Three of the four city interview participants were not aware of the two Supreme Court decisions relating to essential nexus or rough proportionality—*Nollan v. California Coastal Commission* and *Dolan v. City of Tigard*. The third indicated a marginal familiarity based on a developer referencing one of the Supreme Court decisions to be released from mitigation responsibility.

Consultant Practices

Who Pays for Left-Turn Treatment

The two interview participants from consulting firms indicated the developer of a new development would be responsible for paying for the installation of a left-turn treatment at a **site** driveway.

For an off-site improvement, such as an intersection that is projected to be impacted by a new development (but not a site driveway), the following are the responses to the question regarding who would be responsible for paying for the installation of a left-turn treatment:

- The developer or government agency is responsible, on a case-by-case basis.
- The developer is responsible, but sometimes a fair share is paid based on projected impacts.

The following are the responses to the question regarding who would obtain the property from a third party for the construction of a left-turn treatment:

- It is determined on a case-by-case basis and the willingness of the owner to sell to a developer. Owner willingness is typically a determining factor since only governmental agencies can use eminent domain.
- Usually the developer would try to obtain the property, but if the developer is unsuccessful, the government agency with jurisdiction could get involved.

Both of the consultants indicated that the costs for left-turn accommodations are not treated differently than other mitigation needed to handle projected site-generated traffic.

Policy

Both consultants participating in the survey indicated they were aware of no major changes made in the past 5 years related to the decision-making process for the installation of left-turn lanes or other accommodations at unsignalized intersections.

Legal

One of the two consultant interview participants was not aware of the two Supreme Court decisions relating to essential nexus or rough proportionality—*Nollan v. California Coastal Commission* and *Dolan v. City of Tigard*. The second indicated a limited familiarity.

POTENTIAL FUTURE APPLICATIONS (QUESTIONS 23 TO 25)

State DOT Practices

The following are the responses related to lessons learned at the state DOT level that will guide future installations of left-turn accommodations at unsignalized intersections:

- It is easier to install a left-turn lane as part of a project than it would be in the future as a retrofit.

- The application of a left-turn lane is preferred to the usage of a bypass lane.
- Decisions should be made based on the corridor and not only on the specific location under study.
- Left-turn warrants need to be updated.
- Left-turn lanes should be installed even where left-turn volumes may be low.
- Geometric constraints often dictate the dimensions of left-turn lanes.
- DOTs would like to implement more positive left-turn offset.

Generally, state DOT representatives indicated they were either unaware of or not considering changes to their policies related to left-turn accommodations at unsignalized intersections. Three interview participants indicated they either were initiating the process or in the long-term process to update their policies.

State DOT representatives generally indicated they were either unaware of or not considering policy/regulatory changes at other levels of government related to left-turn accommodations at unsignalized intersections. One respondent noted the risk of going this route since it would provide legislators the opportunity to change what is in effect already.

County Practices

The following are the responses related to lessons learned at the county level that will guide future installations of left-turn accommodations at unsignalized intersections:

- Counties need to address issues of safety, capacity, and feasibility, including right-of-way and environmental constraints.
- There are pitfalls to allowing bypass lanes at four-way intersections and problems related to direct driveway access downstream of a bypass lane.
- Counties need to get rid of TWLTLs and put in medians to manage access.
- Counties need to be uniform in application of standards for a development.

County representatives generally indicated they were not considering changes to their policies related to left-turn accommodations at unsignalized intersections. One indicated the county plans to review the existing guidelines as part of an upcoming transportation plan revision.

County representatives generally indicated they were not aware of policy/regulatory changes at other levels of government related to left-turn accommodations at unsignalized intersections. One respondent noted the ability to charge “impact fees” to developers could change how such projects are handled.

City Practices

Three of the four city representatives surveyed did not provide a response to this question. The fourth responded that, based on implementing a few designs, he is more comfortable with identifying in what situations these treatments might be most effective.

The city representatives indicated they were not considering changes to their policies related to left-turn accommodations at unsignalized intersections. They also replied that they were not aware of policy/regulatory changes needed at other levels of government.

Consultant Practices

The two consultants that participated in the interviews did not provide any information related to these survey questions.

APPENDIX F

LEGAL REVIEW

THE IMPACT OF ESSENTIAL NEXUS AND ROUGH PROPORTIONALITY ON DEVELOPMENT CONDITIONS: A LEGAL REVIEW− "ONE THING IS CLEAR: THERE IS STILL A FUNDAMENTAL LACK OF CLARITY"

This chapter addresses the following question: When a government seeks to fulfill a broad public objective such as safety and, in this project, left-turn accommodation, who should bear the costs—the developer who would be adding traffic to the roadway network or the general public? One of the critical issues that needs to be addressed in order to respond to this question is determining the application of the U.S. Supreme Court and other jurisdictional holdings on the takings clause as it may relate to development of property, in particular to determine under what circumstances the requirements of essential nexus and rough proportionality must be applied. The analytical framework presented in this chapter to address these issues attempts to make sense out of court holdings that are often in conflict, erratic, and confusing. In Part I, this chapter begins by looking at the big picture of land use/takings as articulated in the seminal cases of *Nollan v. California Coastal Commission*[1] and *Dolan v. City of Tigard*.[2] This part provides an overview of the development of the legal theories of essential nexus and rough proportionality. Part II attempts to discern the scope of what the Supreme Court meant in laying out guidelines for examining development exactions, and then focuses on small, individual pieces of the puzzle by exploring how essential nexus and rough proportionality have been applied to development exaction in various cases on the federal and state level.

PART I. EXPLANATIONS OF THE LEGAL THEORIES OF ESSENTIAL NEXUS AND ROUGH PROPORTIONALITY

A rudimentary review of the general principles of the takings doctrine as applied to regulatory takings[3] is necessary to evaluate potential applications to left-turn accommodations and to describe the contradictory judicial milieu in which various jurisdictions find themselves. The Fifth Amendment to the United States Constitution provides that no land may be taken for public use without just compensation nor shall private property be taken for public use without just

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[1] *Nollan v. Cal. Coastal Comm'n*, 483 U.S. 825 (1987).
[2] *Dolan v. City of Tigard*, 512 U.S. 374 (1994).
[3] Although outside the scope of this research, note should be made of a relatively recent U.S. Supreme Court case that negated a previous case, *Agins v. City of Tiburon*, 447 U.S. 255 (1980), on regulatory takings issues. In *Lingle v. Chevron U.S.A. Inc.*, 544 U.S. 528 (2005), the Supreme Court noted the imprecision of Agins and spelled out four types of regulatory takings: "Twenty-five years ago, the Court posited that a regulation of private property 'effects a taking if [it] does not substantially advance [a] legitimate state interest' [*Agins*, 447 U.S. at 260]. The lower courts in this case took that statement to its logical conclusion, and in so doing, revealed its imprecision. Today we correct course. We hold that the 'substantially advances' formula is not a valid takings test, and indeed conclude that it has no proper place in our takings jurisprudence. In so doing, we reaffirm that a plaintiff seeking to challenge a government regulation as an uncompensated taking of private property may proceed under one of the other theories…by alleging a 'physical' taking, a Lucas-type 'total regulatory taking,' a Penn Central taking, or a land-use exaction violating the standards set forth in Nollan and Dolan," *Lingle*, 544 U.S. at 540.

compensation.[4] However, there has been a dramatic expansion of the definition of taking as related to the Fifth Amendment. The original focus was on *per se* takings. Federal governmental action unequivocally violated the takings clause if such action resulted in the permanent physical occupation of property or if the action denied the owner all economically beneficial use of her or his property.[5] These actions triggered the "just compensation" requirement no matter whether the invasion was minor or whether the public purpose was greatly served.[6] Thus, the Takings Clause protected private property from physical appropriation by the federal government.[7]

The first expansion of the Takings Clause came in 1922 when the Supreme Court gave "birth to our regulatory takings jurisprudence."[8] In *Pennsylvania Coal Co. v. Mahon*,[9] Justice Holmes stated that "if regulation goes too far, it will be recognized as a taking."[10] Overly burdensome regulations, then, will be seen as a taking. What constitutes "overly burdensome," however, was not defined by Holmes. Instead, he noted that it would be a question of degree.[11] This lack of clarity and formulaic framework continued until the Supreme Court finally began to develop specific tests to determine when, in order to improve the public condition, the regulation went too far. Unfortunately, these attempts only resulted in more confusion. Today, the courts may use one of several tests: a two-pronged inquiry, the Nollan/Dolan rule; a three-part test, the Penn Central rule;[12] and a *per se* rule, referred to as the Lucas rule.[13] For purposes of this study, the focus is on the two-pronged test that was espoused in *Nollan v. California Coastal Commission*[14] and *Dolan v. City of Tigard*.[15]

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[4] U.S. Const. Amend. V.
[5] See *Lucas v. S.C. Coastal Council*, 505 U.S. 1003, 1015 (1992).
[6] Id.
[7] See, e.g., Robert Meltz et al., The Takings Issue: Constitutional Limits on Land Use Control and Environmental Regulation 129–30 (Island Press, 1999); J. Peter Byrne, Ten Arguments for the Abolition of the Regulatory Takings Doctrine, 22 Ecology L.Q. 89, 91–96 (1995); Joseph Sax, Takings and the Police Power, 74 Yale L.J. 36, 58–60 (1964); William M. Treanor, The Original Understanding of the Takings Clause and the Political Process, 95 Colum. L. Rev. 782 (1995); William M. Treanor, The Origins and Original Significance of the Just Compensation Clause of the Fifth Amendment, 94 Yale L.J. 694 (1985).
[8] See, e.g., *Tahoe-Sierra Preservation Council, Inc. v. Tahoe Reg'l Planning Agency*, 535 U.S. 302, 325 (2002) (describing *Pennsylvania Coal Co. v. Mahon,* 260 U.S. 393 (1922) as the case "that gave birth to our regulatory takings jurisprudence." However, some scholars assert that the Court's regulatory takings jurisprudence began in the 19th century with the Court's decision in *Yates v. Milwaukee*, 77 U.S. (10 Wall.) 497 (1870). See, e.g., Kris W. Kobach, The Origins of Regulatory Takings: Setting the Record Straight, 1996 Utah L. Rev. 1211, 1267–72 (1996).
[9] *Pennsylvania Coal Co. v. Mahon,* 260 U.S. 393 (1922).
[10] Id. at 415.
[11] Id. at 416 (stating that "[w]e are in danger of forgetting that a strong public desire to improve the public condition is not enough to warrant achieving the desire by a shorter cut than the constitutional way of paying for the change. As we already have said, this is a question of degree—and therefore cannot be disposed of by general propositions."
[12] See *Penn Cent. Transp. Co. v. City of New York*, 438 U.S. 104, 124 (1978). "The factors include: the economic impact of the regulation on the claimant; the extent to which the regulation has interfered with distinct investment-backed expectations; and the character of the governmental regulation." Id.
[13] *Lucas v. S.C. Coastal Council*, 505 U.S. 1003, 1026–27 (1992). This case held that a regulatory taking occurs where a regulation completely devalued the land, that is, whether the regulations "deprived the land of all economically beneficial use. Id at 1015. See also *supra* note 3.
[14] *Nollan, supra* note 1.
[15] *Dolan, supra* note 2.

Nollan v. California Coastal Commission

In the first case, the Nollans owned a beachfront lot that was located between two public areas, a beach and a park. Between the beach and the rest of the property was an 8-ft-high sea wall. Originally, the Nollans had leased the property with an option to buy; the option was conditioned on their promise to replace an existing house on the lot. The Nollans were required to and did seek the approval of the Coastal Commission to build the new, three-bedroom house on the site of the previous, substantially smaller house. The Commission recommended that the permit be granted on the condition that the Nollans provide a public easement for access across a portion of their property to protect the public's "visual access" to the beach as well as prevent a psychological perception that there was no public access to the beach[16] since the new house would be significantly larger. The Nollans argued that this easement constituted a taking.[17]

The U.S. Supreme Court, with Justice Scalia writing for the majority, stated that the Commission could have denied the permit outright if it would not have "interfered so drastically with the Nollans' use of their property as to constitute a taking" and further a legitimate state interest.[18] The Court reasoned that even though the government required the easement as a condition of the permit—as such, a land use regulation—and even though such land use regulation "does not affect a taking if it 'substantially advance[s] a legitimate state interest' and does not 'deny an owner economically viable use of his land,' "[19] the easement constituted a regulatory taking.[20] In other words, protecting beach access by requiring a permit to build on the beach was a valid exercise of regulation.[21] However, the Commission could not have simply acquired an easement from the Nollans without committing a taking. The Court looked at the connection between the conditions imposed and the interest being protected and held that the easement was not sufficiently related to "visual access": "It is quite impossible to understand how a requirement that people already on the public beaches be able to walk across the Nollans' property reduces any obstacles to viewing the beach created by the new house. It is also impossible to understand how it lowers any 'psychological barrier' to using the public beaches, or how it helps to remedy any additional congestion on them caused by construction of the Nollans' new house."[22] Therefore, the easement condition was invalid since it sought to give the public direct access to the beach rather than focusing on the original condition, visual access: "The evident constitutional propriety disappears, however, if the condition substituted for the prohibition utterly fails to further the end advanced as the justification for the prohibition. When that essential nexus is eliminated, the situation becomes the same as if California law forbade shouting fire in a crowded theater, but granted dispensations to those willing to contribute $100

[16] *Nollan*, 483 U.S. at 827–28.
[17] Id. at 829. For further takings analysis, see, e.g., John A. Humbach, A Unifying Theory for the Just-Compensation Cases: Takings, Regulation, and Public Use, 34 Rutgers L. Rev. 243, 254–62 (1982); Richard A. Epstein, Takings: Private Property and the Power of Eminent Domain 331–33 (1985); Andrea L. Peterson, The Takings Clause: In Search of Underlying Principles Part I—A Critique of Current Takings Clause Doctrine, 77 Cal. L. Rev. 1301, 1301 (1989); and Andrea L. Peterson, The Takings Clause: In Search of Underlying Principles Part II—Takings as Intentional Deprivations of Property without Moral Justification, 78 Cal. L. Rev. 53, 55 (1990).
[18] *Nollan,* 483 U.S. at 836.
[19] Id. at 834.
[20] *Nollan*, 483 U.S. at 841–42.
[21] Id. at 834–35.
[22] *Nollan,* 483 U.S. at 838–39.

to the state treasury."[23] Thus, there would be no taking as long as the development condition furthered the same purpose as the development ban: the exaction—the easement—did not advance the same purpose as the development prohibition—psychological blocking of the view and, in turn, the perception of blocking access to the beach—because the exaction's purpose lacked an "essential nexus"—that is, an essential connection—to the harm the building would cause.[24] Important to this holding is the underlying principle for the essential nexus test, that of the unconstitutional conditions doctrine, implicitly invoked in *Nollan,*[25] which states that *the government cannot condition receipt of a benefit on the applicant's foregoing a constitutional right.* Therefore, even if the government is not constitutionally required to grant a particular privilege or benefit, once it offers that benefit, it may not condition the offer upon the recipient's surrender or waiver of a constitutional right.[26]

Nollan left unanswered questions about the degree of relationship the government must prove for an exaction to be judicially sustained. Still, in practical terms, state and local governmental bodies must establish an essential nexus when implementing exactions.[27] This became explicit in the subsequent exaction case, *Dolan v. City of Tigard*.

Dolan v. City of Tigard

The Dolan case involved the practice of zoning in relation to property rights. In that case, the U.S. Supreme Court refined its analysis of an exaction by looking at the degree of exaction in relation to the burden caused by the development.[28] The holding in the case established limits on the ability of governmental agencies to use land-use regulations to require property owners to make unrelated public improvements. As background, Dolan owned and operated a plumbing store in Tigard, Oregon. She applied for a land-use variance in order to expand the store and pave its parking lot. The city planning commission granted only conditional approval: Dolan was required to dedicate land along an adjacent creek and develop a pedestrian and bicycle pathway to relieve traffic congestion.

Dolan asserted that the requirements were not related to the proposed development and thus constituted an uncompensated taking. The Supreme Court held that a government agency "may not require a person to give up a constitutional right—here the right to receive just compensation when property is taken for public use—in exchange for a discretionary benefit conferred by the government where the benefit sought has little or no relationship to the property."[29] The Court applied a two-prong test. The first prong, established in *Nollan*, must be to determine whether an "essential nexus" exists between the legitimate state interest to be advanced by the restriction on development and the condition exacted by the government.[30] The Court found that the purpose

[23] Id. at 837–39.
[24] Id. at 837 (quoting *J.E.D. Assoc. v. Atkinson*, 432 A.2d 12, 14–15 [N.H. 1981]).
[25] See *Nollan,* 483 U.S. at 836–37; see also Kathleen M. Sullivan, Unconstitutional Conditions, 102 Harv. L. Rev. 1413, 1463 (1989).
[26] See, e.g., Epstein, Unconstitutional Conditions, State Power, and the Limits of Consent, 102 Harv. L. Rev. 4, 6–7 (1988).
[27] See 483 U.S. at 836–37.
[28] See *Dolan*, 512 U.S. at 374.
[29] *Dolan,* 512 U.S. at 385.
[30] *Nollan*, 483 U.S. at 837.

of the permit conditions fell within the legitimate state interest, that of protecting against flooding and preventing congestion, both interests of which would have been served had the development permit application been denied.[31] Then, the Court added a second prong to the analysis. A determination must be made of whether there is a "rough proportionality" between the legitimate interest the government asserts and the actual impact on the landowner's proposed property use, that is, rough proportionality between the costs or harm the development would impose and the cost imposed by the exaction on the developer.[32] Citing the "reasonable relationship" test adopted by a majority of state courts, the Court held that although it did not adopt the phrase itself because it can too easily be confused with the term "rational basis" used to describe the minimal level of scrutiny under the Equal Protection Clause of the 14th Amendment, "a term such as 'rough proportionality' best encapsulates what we hold to be the requirement of the Fifth Amendment. No precise mathematical calculation is required, but the city must make some sort of individualized determination that the required dedication is related both in nature and extent to the impact of the proposed development."[33] Unfortunately, the proportionality requirement of *Dolan* was left ill defined. Interestingly, in a footnote the majority stated that determination of rough proportionality lies with the municipality.[34] In this case, the city satisfied the nexus test but provided insufficient findings to demonstrate rough proportionality. Thus, governmental entities must make individualized determinations to justify the regulations they impose upon developers to ascertain "rough proportionality."

PART II: REVIEW OF OTHER LAW AS APPLIED TO DEVELOPMENT CONDITIONS IN LIGHT OF NOLLAN/DOLAN

The question in this study is: from a legal perspective, when a government seeks to fulfill a broad public objective such as safety and in this project, left-turn accommodation, who should bear the costs—the developer who would be adding traffic to the roadway network or the general public? If the developer should shoulder the cost, frequently used alternatives include the development exaction and the development agreement, the latter of which may include an exaction. What the case law reveals, however, is a history of adjudicating outcomes that are erratic, conflicting, and inconsistent as will be seen in the following discussion.

Government use of development exactions is not a recent phenomenon. However, due to a variety of factors in the last half century including urban sprawl, local governments have increasingly relied upon exactions to finance new development projects. Cities and towns use development exactions to offset the public burden of new development, with developers paying their fair share of public costs generated.[35] In development exactions, a property owner is forced to relinquish something of value, e.g., land or money, in exchange for a building permit. Thus, development exactions occur when a local governmental entity conditions the grant of a development permit on the developer agreeing to pay money, provide materials or services, or dedicate land.[36] The cash payment form of an exaction is typically referred to as a monetary

[31] Id.
[32] *Dolan*, 512 U.S. at 391.
[33] Id.
[34] Id. at 391 n. 8.
[35] See Donald G. Hagman, Public Planning and Control of Urban and Land Development 904 (2d ed. 1980).
[36] See, e.g., Michael H. Crew, Development Agreements after Nollan v. California Coastal Commission, 483 U.S. 825 (1987), 22 Urb. Law. 23, 23–24 (1990).

exaction. One type of monetary exaction is the impact fee, a one-time financial assessment imposed as a condition of development[37] that offsets the municipality's capital expenditures used to construct public off-site infrastructure directly connected with or required because of the new development, for example. The fee is usually determined by legislatively adopted rates.[38] Using this alternative, municipalities hope to avoid triggering the Nollan/Dolan level of scrutiny. However, even though a municipality uses monetary exactions based on legislative enactment rather than adjudicative, discretionary determinations, these alternatives may still be subjected to the Nollan/Dolan tests.

The various development agreements that occur as the result of negotiations between a developer and the local agency appear to be effective and typically are not subject to the Nollan/Dolan mandates.[39] The purpose of these agreements is to expressly limit municipalities from applying new requirements, ordinances, or other land-use changes to ongoing developments.

The basis for development agreements is found in contractual law; they are voluntary, and that suggests that the agreed-upon terms are binding: "The developers…bargain out of their own choice, and municipalities should be able to exact as much as they can…."[40] Courts have recognized the benefits of development agreements. For example, in *Queen Anne's Conservation, Inc. v. County Comm'rs of Queen Anne's County*,[41] the Maryland Court noted that the purpose of the development agreement "is to vest development rights in the landowner or developer in exchange for the dedication and funding of public facilities."[42] Further, the municipality is not "granting the landowner the right to develop nor imposing conditions on such development"[43] but rather "is promising to protect the developer's investment by not enforcing any subsequent land use regulation that may burden the project."[44] The commitment by a developer that adequate infrastructure to serve the development project will be in place in exchange for a guarantee that a developer may proceed for a specified period of time is beneficial to both parties. These agreements, however, must be based on specific legislative authorization—enabling legislation—that allows municipalities to negotiate these agreements. Such legislation should, for example, establish "minimum procedural requirements for the

[37] See Ronald H. Rosenberg, The Changing Culture of American Land Use Regulation: Paying for Growth with Impact Fees, 59 SMU L. Rev. 177, 205–06 (2006).
[38] Id. at 205.
[39] Daniel J. Curtin, Jr., and Jonathan D. Witten, Windfalls, Wipeouts, Givings, and Takings in Dramatic Redevelopment Projects: Bargaining for Better Zoning on Density, Views, and Public Access, 32 B.C. Envt'l Aff. L. Rev. 325, 340–41 (2005).
[40] Catherine Lockhard, Note, Gaining Access to Private Property: The Zoning Process and Development Agreements, 79 Notre Dame L. Rev. 765, 786 (2004).
[41] 855 A.2d 325 (Md. 2004).
[42] Id. at 327. See also Lockhard, *supra* note 40 for a discussion of the *Queen Anne's Conservation* case.
[43] Lockard (citing David L. Callies and Julie A. Tappendorf, Unconstitutional Land Development Conditions and the Development Agreement Solution: Bargaining for Public Facilities after Nollan and Dolan, 51 Case W. Res. L. Rev. 663, 695 [2001]).
[44] Id.

consideration and adoption of development agreements, and spell out the legal effects of such agreements with regard to subsequent land use regulations."[45]

California was the first state to promulgate development agreement legislation.[46] At least a dozen other states have put statutes on their books to authorize local governments to enter into development agreements.[47] According to one commentator, as it now stands, the majority of jurisdictions "do not subject development agreement cases to regulatory takings analysis…. Until the Supreme Court decides a development agreement case, municipalities will likely continue to use development agreements to exact more than would be available with development conditions."[48] Thus, the question of "who pays" would be answered according to the negotiation between the municipality and the developer. Because there is actual bargaining, exactions and the terms thereof will be as stringent as the parties allow through their negotiation.

As the use of various types of development exaction fees expanded, in other jurisdictions litigants looked to the judicial system to determine the constitutionality of these regulatory takings. In response, the U.S. Supreme Court departed from its long tradition of deference to state police power actions and limited local governments' constitutional ability to impose development conditions. In the Nollan/Dolan decisions discussed in Part I, the Court created a heightened scrutiny of development exactions.[49] The Court did not explicitly provide in those cases an answer to the question of whether monetary exactions are subject to the Nollan/Dolan heightened scrutiny. However, a few days after the Dolan decision, a case addressing the issue of whether monetary exaction must be subjected to the Nollan/Dolan requirements was granted certiorari. In this case, *Ehrlich v. City of Culver City*,[50] a property owner and developer, Ehrlich, requested from the city permission to construct a condominium complex on land he already owned and on which he had operated recreational facilities. Concerned about the loss of the recreational facility, the city approved the plaintiff's application conditioned upon payment of a $280,000 fee to be used for additional public recreational facilities.[51] The landowner challenged this monetary exaction as an unconstitutional taking. Ultimately, the Court remanded the case to the U.S. Court of Appeals for the Sixth Circuit for further consideration. On remand, the Court of Appeals reaffirmed its ruling in favor of Culver City. This caused the Supreme Court of California to grant review.[52] California's Supreme Court reversed. The Court noted that the issue was properly analyzed within the statutory framework of California's Mitigation Fee Act. The Court then held that the city met its burden of showing an essential nexus between the permit

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[45] Michael B. Kent, Jr., Forming a Tie That Binds: Development Agreements in Georgia and the Need for Legislative Clarity, 30 Environs Envtl. L. & Pol'y J. 1, 31 (2006).
[46] Cal. Gov't Code 65,865 (West 1997).
[47] See, e.g., Lockard *supra* note 40 (citing Ariz. Rev. Stat. Ann. 9-500.05 [West 1996 and Supp. 2002] [amended 1997]; Colo. Rev. Stat. Ann. 24-68-101 [West 2001]; Fla. Stat. Ann. 163.3220 [West 2000]; Haw. Rev. Stat. Ann. 46-123 [Michie 2001]; Idaho Code 67-6511A [Michie 2001]; La. Rev. Stat. Ann. 33:4780.22 [West 2002]; Nev. Rev. Stat. Ann. 278.0201 [Michie 2002]; N.J. Stat. Ann. 40:55D-45 [West 1991]; Or. Rev. Stat. 94.504 [2001]; S.C. Code Ann. 6-31-10 [Law. Co-op. Supp. 2002]; Va. Code Ann. 15.2-2303.1 [Michie 2003]; Wash. Rev. Code Ann. 36.70B.170 [West 2003]).
[48] Id. (citing John J. Delaney, Development Agreements: The Road from Prohibition to "Let's Make a Deal!", 25 Urb. Law. 49, 55 [1993]).
[49] See *Dolan*, 512 U.S. at 394–96*; Nollan*, 483 U.S. at 838–39.
[50] *Ehrlich v. City of Culver City*, 512 U.S. 1231 (1994).
[51] Id. at 434–35.
[52] *Ehrlich v. City of Culver City*, 911 P.2d 429, 433 (Cal. 1996).

condition and the public impact of the proposed development by demonstrating the connection between the rezoning necessary to construct the condominium complex and the imposition of the fee that would be expended in support of recreational purposes, as a means of mitigating that loss:

> In our view, the intermediate standard of judicial scrutiny formulated by the high court in Nollan and Dolan is intended to address just such indicators [leveraging] in land use "bargains" between property owners and regulatory bodies—those in which the local government conditions permit approval for a given use on the owner's surrender of benefits which purportedly offset the impact of the proposed development. It is in this paradigmatic permit context—where the individual property owner-developer seeks to negotiate approval of a planned development—that the combined Nollan and Dolan test quintessentially applies.[53]

After carefully reviewing the Nollan/Dolan decisions as well as state court decisions, however, the Court reasoned:

> Under this view of the constitutional role of the consolidated "essential nexus" and "rough proportionality" tests, it matters little whether the local land use permit authority demands the actual conveyance of property or the payment of a monetary exaction. In a context in which the constraints imposed by legislative and political processes are absent or substantially reduced, the risk of too elastic or diluted a takings standard—the vice of distributive injustice in the allocation of civic costs—is heightened in either case.[54]

The Court held that "the city failed to show the required rough proportionality between the magnitude of the fiscal exaction and the effects of the proposed development, and the court therefore remanded for further findings."[55] In other words, the record was insufficient to sustain the city's assertion that Ehrlich should pay a mitigation fee of $280,000 as a condition for approval of his request. Ultimately, the Court looked at whether the exaction imposed arose from legislatively formulated development assessments that would, therefore, deserve deference and be subject to a lesser standard of scrutiny than Nollan/Dolan requirements because "the heightened risk of the 'extortionate' use of the police power to exact unconstitutional conditions is not present."[56] Nollan/Dolan requirements would apply, then, if the exaction was pursuant to a discretionary administrative condition.

In particular, the applicability of Nollan/Dolan to adjudicative versus legislative governmental actions is troubling. One commentator notes that it is possible to separate legislation into two categories: "1) legislation that gives a municipality discretion in applying it built into the very language of the ordinance; and (2) legislation that allows for no discretion and is, by default,

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[53] Id. at 438.
[54] Id. at 444.
[55] Id.
[56] Id.

more even-handed in nature."[57] The commentator looked at two cases in the same Oregon Court of Appeals, *Rogers Machinery, Inc. v. Washington County*[58] and *Dudek v. Umatilla County*[59] to "illustrate the dichotomy between the different types of legislation." In the first case, *Rogers Machinery*, the court refused to apply the Nollan/Dolan requirement to a traffic impact fee (TIF) assessed against developments:

> Indeed, nearly all proposed development that conceivably would burden the street and arterial infrastructure within the county is subject to the mandatory fee. Calculation of the fee is, likewise, nondiscretionary. To be sure, because different uses vary in the burden they place on street and arterial infrastructure, the legislation uses a classification scheme to adjust the fee based on those differences. Such classification is common for any number of generally applicable legislative tax, fee, and other assessments; legislative classifications do not render the scheme adjudicatory or discretionary. The TIF ordinance is, in short, precisely the kind of detailed and uniformly applied legislative assessment scheme that courts in other states hold does not fall within the express reach or the implicit rationale of Dolan's heightened scrutiny test.[60]

A similar challenge was at issue in *Dudek*, a case involving application of an ordinance to road widening. This time, the court applied the Nollan/Dolan analysis because there was an increased risk that the governmental entity could abuse its discretion in applying the legislation.[61] The court reasoned:

> As we discussed in Rogers Machinery, Inc., a significant consideration in the determination of whether Dolan's rough proportionality test applies to a particular government action is whether the action is taken pursuant to a legislatively adopted scheme that applies to a broad class of property and whether the action involves the exercise of discretion. Also pertinent is whether the government action requires adjudication to determine whether and how a government regulation applies to particular property. As we explained in Rogers Machinery, Inc., one of the main reasons those considerations are pertinent is that, when government action is taken pursuant to a legislatively adopted standard that was not adopted to apply to particular properties or development but rather to a broad class of property, and when there is no need for adjudication or the exercise of discretion at the time that the standard is applied to a particular property, there is far less danger of a governmental entity attempting to use its power to extort unconstitutional conditions from persons seeking governmental approval of a specific proposal…. [T]he practical reality is that application of this ordinance to

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[57] Jane Needleman, Exploring Exactly When Nollan and Dolan Should be Triggered, 28 Cardozo L. Rev. 1563 (2006).
[58] *Rogers Machinery, Inc. v. Washington County*, 45 P.3d 966 (Or. Ct. App. 2002).
[59] *Dudek v. Umatilla County*, 69 P.3d 751 (Or. Ct. App. 2003).
[60] Id. at 982.
[61] See Needleman, *supra* note 57.

> a particular case requires a significant exercise of discretion. It effectively requires an adjudication in each case.[62]

Thus, the court focused on whether there was potentially a significant amount of discretion even if the ordinance was legislative. Common sense would dictate that a court would find it difficult to look through the layers of broad, discretionary application of legislative action and instead would prefer clear-cut delineations of application. For those courts that distinguish *Dolan* on this basis, it is the assumption that *Dolan* applies to adjudicative, discretionary regulations.

Other state courts, though, including Illinois, Ohio, Oregon, Texas, and Washington apply the Nollan/Dolan heightened scrutiny to monetary exactions.[63] As well, some state courts have determined that impact fees, regardless of whether they came about through legislation or other means, trigger the Nollan/Dolan heightened scrutiny analysis. For example, in the case of *Town of Flower Mound v. Stafford Estates Ltd. P'ship,*[64] the Texas Supreme Court, in deciding whether there was a compensable taking where a town conditioned its approval of the development of a residential subdivision on the developer's rebuilding an abutting road, noted:

> Conditioning government approval of a development of property on some exaction was a compensable taking unless the condition: (1) bore an essential nexus to the substantial advancement of some legitimate government interest and (2) was roughly proportional to the projected impact of the proposed development. There was no important distinction between a dedication of property to the public and a requirement that property already owned by the public be improved. The town failed to relate discounted traffic impact fees to the impact of developments on traffic. Conditioning development on rebuilding the road with concrete and making other changes was simply a way for the town to extract from the developer a benefit to which the town was not entitled. The exaction was thus a taking for which the developer was entitled to be compensated.[65]

In *Home Builders Ass'n of Dayton & the Miami Valley v. City of Beavercreek,*[66] the Supreme Court of Ohio addressed the issue of whether an ordinance that allows impact fees payable by developers of real estate to aid in the cost of new roadway projects is constitutional. The Court had consistently held that under specific legislative language, municipalities had the authority to impose exactions "provided that the municipality is not statutorily forbidden from doing so, and the exactions meet constitutional standards."[67] The Court reviewed Nollan/Dolan requirements, the requirements noted in other states' court opinions, and those articulated in its own decisions and held that the impact fee ordinance did not violate the Ohio Constitution or the United Station Constitution.

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[62] 69 P.3d at 756.
[63] See, e.g., Daniel J. Curtin, Jr., and W. Andrew Gowder, Jr., Exactions Update: When and How Do the Dolan/Nollan Rules Apply?, 35 Urb. Law. 729, 733–38 (2003).
[64] *Town of Flower Mound v. Stafford Estates Ltd. P'ship*, 135 S.W.3d 620, 641 (Tex. 2004).
[65] Id.
[66] *Home Builders Ass'n of Dayton & the Miami Valley v. City of Beavercreek,* 729 N.E.2d 349, 356 (Ohio 2000).
[67] Id.

In *Benchmark Land Co. v. City of Battle Ground*,[68] the developer applied to the City of Battle Ground for a development permit. As a condition of approving the application, the City required the developer to make half-street improvements to a street adjoining the development. The Court applied the Nollan/Dolan analysis to the condition and held that the City failed to show that the condition was proportional to the development's impact on the street. The Court noted that "the condition advanced a legitimate state interest, improving the public roads, and the condition did not deny the developer all economically viable use of its land…. The City did not restrict development of property, but required developer to address a problem that existed outside the development property, an adjoining street in need of improvement. There was a necessary connection between the condition and the public problem."[69]

Courts in Arizona, Colorado, Kansas, and Maryland have held that Nollan/Dolan standards do not apply to monetary exactions. For example, in *Krupp v. Breckenridge Sanitation Dist*.,[70] a case heard by the Supreme Court of Colorado, the developers of a townhouse project contended that the required "plant investment fee" (PIF) that was assessed for their project was an unconstitutional taking. The Court found that the PIF was "a valid, legislatively established fee that was reasonably related to respondent district's interest in expanding its infrastructure to account for new development, and that [the Sanitation District's] specific PIF assessment on [the developer's] project was fairly calculated and rationally based. As such, the PIF did not fall into the narrow category of charges that were subject to a Nollan/Dolan takings analysis."[71] The distinction, then, was that the PIF was legislatively mandated as opposed to an adjudicative, discretionary determination. This distinction has played a crucial role in several of the cases already discussed, and the courts have taken note of the *Ehrlich* rationalization in which a greater threat of extortion exists with discretionary determinations. The Court also held that the PIF was "purely a monetary assessment rather than a dedication of real property for public use."[72]
In *Homebuilders Ass'n of Central Arizona v. City of Scottsdale*,[73] the Arizona Supreme Court focused on the legislative versus adjudicative distinction and held that the ordinance in question was legislative act and therefore "came to the court cloaked with a presumption of validity…. Land use regulations of general application will be overturned by the courts only if a challenger shows the restrictions to be arbitrary and without a rational relation to a legitimate state interest…. Development or impact fees are presumed valid as exercises by legislative bodies of the power to regulate land use."[74] The Court held that *Dolan* was inapplicable for two reasons. First, the reasonableness of the amount of the fee was not an issue, so no question arose as to whether the fee was roughly proportional to the burden imposed on the community; therefore, a *Dolan* test was not applicable.[75] Second, *Dolan* is distinguishable because *Dolan* involved a city's adjudicative decision to "impose a condition tailored to the particular circumstances of an individual case" as the Chief Justice took care to explain. The present case involved a legislative

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68 *Benchmark Land Co. v. City of Battle Ground*, 14 P.3d 172, 175 (Wash. Ct. App. 2000).
69 Id.
70 *Krupp v. Breckenridge Sanitation Dist.*, 19 P.3d 687, 695-98 (Colo. 2001).
71 Id.
72 Id. at 697.
73 *Home Builders Ass'n v. City of Scottsdale*, 187 Ariz. 479 (1997).
74 Id.
75 Id.

decision by the city. Although agreeing that the question has not been settled by the U.S. Supreme Court, the court referred to *Ehrlich v. City of Culver City*,[76] stating that the case:

> dramatically illustrates the differences between the two exactions. In Ehrlich, the city had imposed an individually tailored $280,000 mitigation fee as a condition of approving a rezoning request. On remand from the United States Supreme Court for reconsideration in light of Dolan, the California Supreme Court held the record insufficient to show that the fee was roughly proportional to the public burden of replacing recreational facilities that would be lost as a result of rezoning Ehrlich's property. The California court suggested that the Dolan analysis applied to cases of regulatory leveraging that occur when the landowner must bargain for approval of a particular use of its land…. The risk of that sort of leveraging does not exist when the exaction is embodied in a generally applicable legislative decision.[77]

The case was appealed to the U.S. Supreme Court, but the Court denied the writ of certiorari.[78] That is unfortunate because at least two members of the Supreme Court have noted in a dissenting opinion in denying a writ that the legislative versus adjudicative distinction is not valid. The case, *Parking Ass'n of Georgia v. City of Atlanta*,[79] came before the U.S. Supreme Court at about the same time as the *Homebuilders* decision by Arizona's Supreme Court. In *Parking*, the Atlanta City Council passed an ordinance requiring certain existing surface parking lots to include landscaped areas "equal to at least 10% of the paved area and to have at least one tree for every eight parking spaces. The ordinance covers some 350 parking lots; petitioners estimate that compliance with the landscaping requirements will cost approximately $12,500 per lot, for a total of $4,375,000. Additionally, parking lot owners will lose revenue due to lost parking spaces and lost advertising dollars: the trees allegedly will obscure existing advertising signs and cause petitioners to lose contracts worth about $1,636,000."[80]

Justice Thomas, joined by Justice O'Conner, wrote in the dissent denying the writ that a taking may occur when there is a legislative act. Justice Thomas maintained that the case should have been heard:

> It is not clear why the existence of a taking should turn on the type of governmental entity responsible for the taking. A city council can take property just as well as a planning commission can. Moreover, the general applicability of the ordinance should not be relevant in a takings analysis. If Atlanta had seized several hundred homes in order to build a freeway, there would be no doubt that Atlanta had taken property. The distinction between sweeping legislative takings and particularized administrative takings appears to be a distinction without a

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[76] Id. (citing *Ehrlich v. City of Culver City*, 12 Cal. 4th 854, 911 P.2d 429 [Cal. 1996], cert. denied, 136 L. Ed. 2d 218, 117 S. Ct. 299 [1996]).
[77] Home Builders, *supra* note 73 at 486. See also Jonathan M. Block, Limiting the Use of Heightened Scrutiny to Land-Use Exactions, 71 N.Y.U. L. Rev. 1021, 1024 n.154 (1996).
[78] *Homebuilders Ass'n v. City of Scottsdale*, *supra* note 73.
[79] *Parking Ass'n of Georgia v. City of Atlanta*, 264 Ga. 764, 450 S.E.2d 200 (Ga. 1994), cert. denied, 515 U.S. 1116, 132 L. Ed. 2d 273, 115 S. Ct. 2268, 2269 (1995).
[80] Id.

> constitutional difference…. The lower courts should not have to struggle to make sense of this tension in our case law. In the past, the confused nature of some of our takings case law and the fact-specific nature of takings claims has led us to grant certiorari in takings cases without the existence of a conflict.[81] Where, as here, there is a conflict, the reasons for granting certiorari are all the more compelling.[82]

In the federal Circuit Courts, the Ninth and Tenth both apply a narrower Nollan/Dolan standard. In an interesting twist, the Ninth Circuit includes California.[83] Thus, California has two opposing rules, once again demonstrating the confusion and conflict involved in exactions and the regulatory takings issues.

These cases point out that it is clear there is no clarity. Determinations of applicability of the Nollan/Dolan standards will be adjudicated on a case-by-case basis. However, what does seem to reveal some clarity is the fact that if an impact fee or other monetary exaction is legislatively mandated, then it may not fall within the development exactions that triggered the Nollan/Dolan requirements.

How do these oftentimes inconsistent, conflicting, and erratic decisions apply to governmental actions such as providing for safe highways and, in particular, left-turn accommodations? Who should bear the cost when a government seeks to fulfill a broad public objective such as safety and, in this project, left-turn accommodation—the developer who would be adding traffic to the roadway network or the general public?

The answers can apparently only be derived by looking at specific jurisdictions, and even then, as in the case of California and the Court of Appeals in Oregon, there may be conflicting applications. Rather than having the consistency that is anticipated and expected from the judicial system, in the area of development conditions cases, courts appear to engage in ad hoc, individualized, factual determinations of whether a governmental entity has exceeded constitutional limitations. Thus, the burden of proof has shifted so that the government must jump through an additional hurdle; the justification for exaction must be clear. No longer is the presumption of validity of legislatively mandated exactions unassailable.

This burden could result in potentially negative results, for example, for environmentalists who propose strong, enforceable regulations to protect cities and rural areas from expansive development or for those who see that government actions are critical in providing safety for its citizenry. The resolution of this conflict is critical, particularly when growth management schemes are becoming increasingly commonplace with government entities searching for ways to fund such programs. Shifting the cost of development including “smart growth” or “sustainable growth” to property owners through exactions such as impact fees may appear a logical alternative. From the mishmash of court opinions, however, a governmental entity

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[81] Id (citing Tigard, 512 U.S. at ___ [slip op., at 8] observing that certiorari was granted because the Oregon Supreme Court allegedly had misapplied *Nollan v. California Coastal Comm’n*, 483 U.S. 825, 97 L.Ed. 2d 677, 107 S. Ct. 3141 [1987]).
[82] *Parking Ass’n of Georgia v. City of Atlanta*, 515 U.S. 1116.
[83] See, e.g., Lockhard, *supra* note 40 at 742.

should, at the very least, have these fees buttressed by clear legislative language. Who should pay? Unless the facts of the case are clear cut, the outcome is unpredictable even within specific jurisdictions. Results will be more predictable if agencies have the documented authority to manage access. This authority could be achieved in a number of different ways (municipal codes for access management, administrative rules, etc.). Otherwise, there are too many hazy cases and multiple factors so that predicting the outcome is impossible. The U.S. Supreme Court must articulate an applicable rule to bring consistency and predictability in order to answer the question of “who should bear the burden, who should pay.”

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