Design Guidance for Intersection Auxiliary Lanes

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AUTHORS

Fitzpatrick, Kay; Brewer, Marcus A.; Dorothy, Paul; and Eun Sug Park

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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Kay Fitzpatrick
TEXAS A&M TRANSPORTATION INSTITUTE
College Station, TX

Marcus A. Brewer
TEXAS A&M TRANSPORTATION INSTITUTE
College Station, TX

Paul Dorothy
WHITE STAR ENGINEERING CONSULTANTS
Worthington, OH

Eun Sug Park
TEXAS A&M TRANSPORTATION INSTITUTE
College Station, TX

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FOREWORD

By B. Ray Derr
Staff Officer
Transportation Research Board

AASHTO's *A Policy on Geometric Design of Highways and Streets* (the Green Book) contains a limited amount of guidance for auxiliary lanes at intersections. This report expands on that guidance, particularly regarding bypass lanes, channelized right-turn lanes, deceleration and taper length, design and capacity of multiple left-turn lanes, and alternative intersection designs. The report will be very useful in updating agency design manuals and to those designing intersections.

A large proportion of crashes occur at intersections and auxiliary turn lanes are a key countermeasure for addressing such crashes. Auxiliary lanes can also be used to increase capacity and improve operations at an intersection. The design components of a traditional auxiliary turn lane consist of the length needed to store an appropriate number of waiting vehicles, a vehicle deceleration area, and the taper needed to develop the full lane width. Offset and indirect turn lanes and other types of auxiliary lanes (e.g., acceleration lanes) have similar components. The guidance and practice used throughout the United States for auxiliary lane designs and application vary by intersection location (e.g., rural or urban), traffic control (e.g., stop-control or signal-control), and lane type (e.g., right- or left-turn). The AASHTO Green Book contains limited criteria for geometric design of auxiliary lanes at intersections. Additional support for these criteria and expansion of the material to cover additional designs are needed to fully realize the safety and operational benefits of auxiliary lanes at intersections.

In NCHRP Project 03-102, the Texas A&M Transportation Institute reviewed existing literature and ongoing research projects and identified issues meriting further study to validate, enhance, and expand current Green Book guidance. Field studies were conducted to assess the operation of double left-turn lanes and deceleration lanes. The research team then developed practical guidance for designers on auxiliary lanes, including recommendations for improving the Green Book.

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Kay Fitzpatrick, TTI senior research engineer, was the principal investigator. The other authors of this report are Marcus A. Brewer (associate research engineer, TTI), Paul Dorothy (principal, White Star Engineering Consultants), and Eun Sug Park (research scientist, TTI). The work was performed under the general supervision of Dr. Fitzpatrick.

The authors wish to acknowledge the many individuals who contributed to this research by participating in the state-of-the-practice questionnaire, assisting with the review of literature and reference manuals, helping with field data collection and reduction, and assisting with other project-related tasks. Those individuals include (but are not limited to)

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- Mehdi Azimi, graduate student, Texas A&M University.
- Noopur Shah, student worker, TTI.
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CONTENTS

Note: Many of the photographs, figures, and tables in this report have been converted from color to grayscale for printing. The electronic version of the report (posted on the Web at www.trb.org) retains the color versions.

SUMMARY

Design Guidance for Intersection Auxiliary Lanes

Crashes at intersections constitute a large portion of the annual traffic crashes in the United States. Previous experience has shown that auxiliary turn lanes are an effective countermeasure to address some of these crashes, and they are documented as having sizable crash modification factors in the AASHTO *Highway Safety Manual.* Auxiliary lanes can also be used to increase capacity, improve traffic operations, and increase driver comfort at an intersection.

The AASHTO *Policy on Geometric Design of Highways and Streets* (commonly known as the Green Book) contains some guidance for geometric design of auxiliary lanes at intersections, but more information would benefit the profession. The objective of NCHRP Project 03-102 was to recommend improvements to the guidance provided in the AASHTO Green Book for auxiliary lanes at intersections, thereby leading to improved safety and operations. Specifically, the research team sought to provide additional support for existing criteria and expand the material to fill current information needs. The anticipated result was that the additional guidance would help the profession to realize fully the safety and operational benefits of auxiliary lanes at intersections and improve the consistency of application for these fundamental road features.

Researchers reviewed recent literature, state design manuals, and multiple editions of the AASHTO Green Book to determine the state of the practice and the basis for it. To identify practices and evaluations not adequately documented in traditional literature and/or design manuals, the research team interviewed practitioners at state DOTs about current practices and potential guidance needs, based on their professional experience and the policies of their respective departments. The interviews focused on current design practices for deceleration lanes, multiple turn lanes, and channelizing island design. Practitioners listed the following among their key responses:

- Reasons for installing a deceleration lane and/or determining its length vary based on other geometric and traffic considerations, with capacity and speed being among the most common.
- Agencies use numerous sources as the basis for their deceleration lane guidelines, although most sources are based on the AASHTO Green Book and/or *NCHRP Report 279.*
- When the preferred dimensions of a deceleration lane (i.e., taper length, deceleration length, and storage length) cannot be accommodated within a particular site, the decision for how to make adjustments and reductions is typically a qualitative one, although the factors that contribute to that decision are not always well defined.
- Most agencies have some kind of guidance regarding double left-turn lanes, although such guidance may not be very detailed. The decision to install such a treatment is frequently based on the current or expected turning demand, but protected-only signalization is also a typical criterion.

- Guidance on how to adjust the key design dimensions of a double left-turn lane is widely varied, if available. The adjustments for storage, deceleration, and taper of a double left-turn lane are often determined qualitatively or on a case-by-case basis.

As part of the questionnaire sent to key state transportation agencies, respondents were asked to identify locations with installations that would be considered best-practice sites. These best-practice sites were to demonstrate preferred design treatments for five design categories: island design, deceleration lane design, double left-turn lane design, triple left-turn lane design, and double right-turn lane design. Researchers selected a single site for each of the five design categories and examined it in detail through a case study approach. The case studies indicated that the guidance found in the 2011 Green Book is adequate for island design. However, supplemental guidance may be needed to address (1) the design needs of providing deceleration lanes in a flared intersection configuration and (2) clearance distances for multiple turn lanes.

Using the findings from these information-gathering activities, along with input from the project advisory panel, the research team identified issues to consider for further study, the panel then selected two field studies for completion in Phase II:

- Operational study on double left-turn lanes.
- Operational study on deceleration lanes.

The goal of the double left-turn lane operational study was to determine the effects of geometric characteristics on double left-turn lane operations, as measured with saturation flow rate, lane distribution, and driver behaviors. Receiving leg width, left-turn lane width, and downstream friction type and distance were the key geometric variables studied. Identifying sites with the desired range of receiving leg width was the most difficult of the study variables to satisfy, although finding sites with an average double left-turn lane width greater than 12 ft also proved challenging. Data from 26 sites in three states (i.e., Arizona, California, and Texas) were used in the analyses. The data collection method was video recording.

Key findings from the analyses of operations at double left-turn lanes include the following:

- A total of 10,023 saturation flow rate values were available for study. The average double left-turn lane saturation flow rate for these 10,023 data points was 1,775 passenger cars per hour of green per lane (pcphgpl).
- The lane variable was found to be not significant, which means that the inside and outside lane saturation flow rates were similar. The number of vehicles in the queue was also not significant.
- The model found that, for each additional U-turning vehicle within the left-turn queue, saturation flow rate would decrease by 56 pcphgpl.
- The analysis of the effects of the friction point type and location revealed that the analysis needed to include a new variable that accounted for a dedicated lane added at the end of a channelized right turn. Although the turning vehicles were constrained to two lanes at the start of the receiving leg, a review of the video data revealed that drivers in the outside lane would angle their vehicle to make a smooth entry into the new lane. This behavior resulted in higher saturation flow. The model results indicated that the addition of this new lane resulted in an increase in saturation flow rate of about 50 pcphgpl.
- The *Highway Capacity Manual* indicates that wider lane widths are associated with higher saturation flow rate, but this study found that the width of the left-turn lanes did not significantly affect saturation flow. This finding could be construed to mean that narrow lanes can be used without affecting operations. In making this interpretation, however,

a key component of the study design is not represented. The recommended method to determine saturation flow rate requires the elimination of a queue if a heavy vehicle is present within the queue. Therefore, within this study, while the operations of queues with only passenger cars were similar for the various left-turn lane widths studied (9.5 to 13 ft), the operations of queues that include heavy vehicles (trucks or buses) may have different results.

- The width of the receiving leg represents the visual target for the left-turning drivers. For the sites included in this study, receiving leg width ranged from 24 ft to 54 ft, and analysis found that the width affected the saturation flow rate. When the receiving leg width was between 24 and 36 ft, the average saturation flow rate was 1,725 pcphgpl, while a receiving leg width of 40 to 54 ft was associated with an average saturation flow rate of 1,833 pcphgpl.

The goal of the deceleration lane study was to determine the effects of taper length and posted speed limit on approach speeds and deceleration rates of left-turning vehicles, as compared to those described in the Green Book. Researchers collected data from 12 sites in four states (i.e., Alabama, Florida, Mississippi, and Texas). The primary data collection method was video recording, although lidar, global positioning systems (GPS), and traffic counters were used to provide supplemental data.

Following data reduction and quality control checks, data from 410 left-turning vehicles were analyzed to investigate three key deceleration lane design guidelines in the Green Book:

- The speed differential between turning vehicles and following through vehicles is 10 mph when the turning vehicle clears the through-traffic lane.
- The values for deceleration length are based on a 5.8 ft/s^2 average deceleration while moving from the through lane into the left-turn lane.
- The values for deceleration length are based on a 6.5 ft/s^2 average deceleration after completing the lateral shift into the left-turn lane.

Key findings from the analyses of operations at deceleration lanes were

- The Green Book states that a 10-mph differential is acceptable on arterials, but drivers in this study commonly had speed differentials greater than 10 mph, and the higher speed differentials in this study often occurred for vehicles traveling at higher speeds. The Green Book describes the 10-mph differential in relation to a comfortable deceleration for the driver, while support for using 10 mph as the threshold for speed differential may be better stated in terms of reducing the likelihood of a crash.
- The speed differential was significantly and positively related to the upstream speed. Other variables, such as posted speed limit, deceleration length, and taper length, also had various effects on speed differential in the statistical models, and the general effect of deceleration length appears to be consistent with the Green Book text that describes higher differentials with shorter length, but those effects were not statistically significant at the 0.05 level.
- The Green Book deceleration rate of 5.8 ft/s^2 prior to the end of the taper was within the range of average rates at the study sites, but 8 of the 12 sites had an average rate higher than 5.8 ft/s^2. The 50th percentile rate was approximately 6.1 ft/s^2 for low-speed sites and 6.7 ft/s^2 for high-speed sites. In addition, 85% of observed drivers at high-speed sites decelerated at a rate of 4.2 ft/s^2 or greater up to the end of the taper.
- The deceleration length (at low-speed sites), the speed at the upstream counter, and the speed in the taper were all significant in affecting the rate of deceleration prior to the end of the taper.

- A design that accommodates decelerating at 4.2 ft/s^2 during the lateral movement into the turning lane provides for a more gradual, controlled deceleration, but a higher deceleration rate (closer to 6.5 ft/s^2 for half of the observed drivers or 10 ft/s^2 for the most aggressive drivers) could be acceptable if site constraints or other factors dictate a shorter length. The tradeoff for the shorter length, however, would occur in one or both of the following forms:
  - Less aggressive drivers would begin their deceleration earlier, either through coasting or applying the brake further upstream of the beginning of the taper, increasing the speed differential between turning and through vehicles.
  - Some drivers would accomplish more of their deceleration after lateral movement, leading to much higher deceleration rates approaching the stop line and/or the back of the queue.
- Deceleration length and the vehicle speed in the taper were found to be statistically significant in determining deceleration rate within the full-width deceleration lane.
- Compared to the 6.5 ft/s^2 rate noted in the Green Book, approximately two-thirds of the drivers observed making left turns at the study sites decelerated at greater rates to come to a stop at the stop line. Data from this study indicate that a designer could produce a left-turn lane design associated with a deceleration rate of 6.5 ft/s^2 and it would accommodate the current behavior of 85% of left-turning drivers at high-speed sites and half of drivers at low-speed sites.

Researchers developed a list of recommended revisions using the findings from the two field studies, along with the research team's review of the literature and state design manuals. Key recommended changes are as follows:

- Expanding the discussion of bypass lanes in Section 9.3.1, including a cross-reference to warrants suggested for inclusion in a revised Section 9.7.3, based on research in *NCHRP Report 745*.
- Updating the text in multiple sections to better reflect current practice and guidance (including the *Highway Design Handbook for Older Drivers and Pedestrians*) on skew angle, which should not be less than 75 degrees.
- Inserting a new subsection on channelized right-turn lanes in Section 9.6.1 to improve the level of detail found in the Green Book, based on research from NCHRP Project 3-89.
- Revising the section on deceleration length and taper length in Section 9.7.2 to incorporate the findings from this project's Task 4 deceleration field study. This included adding a subsection on Perception-Reaction Distance, replacing Figure 9-48 and Table 9-22 for clarity and consistency, and better defining the purpose, dimensions, and design of a taper used to add a left-turn lane.
- Revising the text in Section 9.7.3 to better describe the capacity benefits and design considerations for multiple left-turn lanes, based on findings from the double left-turn lane operational study in Task 4 of this project.
- Making multiple revisions in Sections 9.8 and 9.9 to better describe design features of alternative intersection designs, such as U-turn crossovers, median openings, and secondary intersection spacing.

CHAPTER 1

Background

Research Problem Statement

There were approximately 5.8 million traffic crashes in 2008, with 55% of them occurring at intersections. Auxiliary turn lanes have been clearly identified as an effective countermeasure to address these crashes, as documented by the sizable crash modification factors for turn lanes in the AASHTO *Highway Safety Manual* (HSM) (*1*). Auxiliary lanes can also be used to increase capacity and improve traffic operations at an intersection.

The AASHTO *Policy on Geometric Design of Highways and Streets* (commonly known as the Green Book) (*2*) contains limited criteria for geometric design of auxiliary lanes at intersections. Additional support for these criteria and expansion of the material to cover additional designs are needed to realize fully the safety and operational benefits of auxiliary lanes at intersections and to improve the consistency of application for these fundamental road features.

Research Objectives

The objective of this research was to recommend improvements to the guidance provided in the AASHTO Green Book for auxiliary lanes at intersections, thereby leading to improved safety and operations.

Research Approach

The research consisted of six tasks. Each task listed is followed by the objectives of that task:

- **Task 1. Conduct Review of Literature and Ongoing Research:** identify the state of design practice for auxiliary lanes at intersections by gathering and synthesizing information on existing (customary and innovative) practices and research.
- **Task 2. Identify Issues:** determine the issues that merit further study to validate, enhance, and expand current Green Book guidance. Task 2 activities included a review of existing state design manuals, interviews with practitioners in state DOTs, and an initial review of Chapter 9 of the Green Book to describe items that are candidates for further study and potential revision.
- **Task 3. Submit Interim Report and Research Plan:** submit the interim report along with the research plan.
- **Task 4. Perform Approved Research Plan:** implement the revised field data collection and analysis plan.
- **Task 5. Develop Recommended Changes to Green Book and Other Guidance Documents:** develop recommended changes to the Green Book and other guidance documents, as appropriate.
- **Task 6. Prepare Final Report:** prepare, in accordance with NCHRP guidelines, this final report.

Report Organization

This final report contains the following chapters and appendices:

- **Chapter 1: Background.** 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: Review of Literature and State Design Guidance.** This chapter presents the findings from a review of relevant literature and a review of design manuals on state DOT websites.
- **Chapter 3: State of the Practice.** As part of Task 2 efforts, the research team conducted a state-of-the-practice review of current design considerations. To accomplish that objective, the research team contacted representatives from a selection of state DOTs to inquire about current practices and potential guidance needs, based on their professional experience and the policies of their respective departments. This chapter describes that review and its findings.

- **Chapter 4: Typical Designs.** As part of the questionnaire sent to state DOTs in Task 2, respondents were asked to identify locations with installations that would be considered best-practice sites. These best-practice sites were to demonstrate preferred design treatments for five design categories: island design, deceleration lane design, double left-turn lane design, triple left-turn lane design, and double right-turn lane design. A single site considered representative of the design treatment was identified for each of the five design categories. This representative site was examined in detail through a case study approach; those case studies are provided in this chapter.
- **Chapter 5: Double Left-Turn Lane Field Study.** The goal of the Task 4 double left-turn lane study was to determine the effects of geometric characteristics on operations, as measured using saturation flow rate (SFR) and lane utilization, for double left-turn lanes. The geometric variables that were the focus of this study were receiving leg width, left-turn lane width, and downstream friction location (type and distance). Chapter 5 describes the activities and findings associated with that study.
- **Chapter 6: Deceleration Field Study.** The objectives of the Task 4 deceleration study were to determine the effects of taper length and posted speed limit on approach speeds and deceleration rates of left-turning vehicles, as compared to those described in the Green Book. This chapter contains the details of that study.
- **Chapter 7: Conclusions, Recommendations, and Suggested Research.** This chapter lists the research team's conclusions from the information obtained in Tasks 1 through 5. It also lists researchers' recommendations for applying the findings and suggestions for future research.
- **Appendix A: Recommended Revisions to AASHTO Green Book.** Appendix A contains the results of the research team's review of the Green Book in conjunction with the findings and conclusions from the research in Tasks 1 through 4. The appendix also contains researchers' recommendations for revisions to consider including in the next edition of the Green Book.

CHAPTER 2

Review of Literature and State Design Guidance

The following sections describe recent research efforts related to design elements and considerations for auxiliary lanes at intersections. Although much of the recent research investigated aspects of left-turn lanes as drivers approach intersections, there were also findings related to right-turn lanes and to acceleration lanes downstream of intersections.

Literature

Warrants

Left-Turn Lane Installation Guidelines

Although many procedures are in use by organizations to determine the need for left-turn lanes, several are either very similar or identical. The oldest research found on evaluating the need for left-turn lanes at unsignalized intersections was that of M. Harmelink (*3*) in a paper 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 stated 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 presented 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 were given, as well as 5, 10, 15, 20, 30, and 40% left-turn volumes.

Many variations of installation guidelines are based on Harmelink's findings. For example, the 2004 edition of AASHTO's *A Policy on Geometric Design of Highways and Streets* (commonly known as the Green Book) (*4*) contains a table (see Table 2-1) for use in determining the need for a left-turn lane on two-lane highways. Similar tables were also present in the 2001, 1994, 1990, and 1984 editions of the Green Book. The values in the tables are based on Harmelink's work.

In 1985, TRB published *NCHRP Report 279: Intersection Channelization Design Guide* (*5*). In that report, data from Harmelink's work were used to establish guidelines for determining the need for a left-turn lane. The following advice was 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 a series of tables.
4. Left-turn lanes on stopped or secondary approaches should be provided based on analyses of the capacity and operations of the unsignalized intersection. Considerations include minimizing delays to right-turning or through vehicles and total approach capacity.

Many state DOTs also use a variation of Harmelink's work in their design manuals, typically through reference to the Green Book or through specifically reproducing Green Book values. A review of state design manuals (*6*) revealed the following:

- Nine state manuals either include the same table of criteria as the values included in the Green Book for determining the need for a left-turn lane or reference the Green Book.
- Three of these states also include information from *NCHRP Report 279* (*5*) or from Harmelink's original paper (*3*).
- Six states include the graphs available in *NCHRP Report 279* along with some of the recommendations, while two more states include most of the recommendations from *NCHRP Report 279*, without referencing the graphs.

Table 2-1. AASHTO installation guidelines for left-turn lanes on two-lane highways (*4*).

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

Source: *A Policy on Geometric Design of Highways and Streets* (2011) by AASHTO, Washington, D.C. Used by permission.

- The New Jersey manual references the Harmelink paper directly.

FHWA's *Signalized Intersections: Informational Guide* (*7*) states that in the absence of site-specific data, the designer should refer to the 2000 edition of the *Highway Capacity Manual* (HCM) (*8*), which indicates the probable need for a left-turn lane if the left-turn volume is greater than 100 vehicles in a peak hour, and the probable need for double left-turn lanes if the volume exceeds 300 vehicles per hour. The HCM also indicates a left-turn lane should be provided if a left-turn phase is warranted. The FHWA *Signalized Intersections: Informational Guide* presents several rule-of-thumb intersection capacities for various scenarios where exclusive left-turn treatments may be required on one or both approaches to an intersection. In general, the guidelines adapted from *NCHRP Report 279* say that exclusive left-turn lanes are needed when a left-turn volume is greater than 20% of total approach volume or when a left-turn volume is greater than 100 vehicles per hour in peak periods (*5*).

NCHRP Project 3-91 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* (*1*), 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 discussed left-turn lane designs, traffic control treatments, and case study examples (*9*). The recommended left-turn treatment warrants developed based on that research from NCHRP Project 3-91 are provided in Appendix A; the three sets of warrants apply to

- Rural two-lane highways (left-turn lanes and bypass lanes).
- Rural four-lane highways (left-turn lanes).
- Urban and suburban roadways (left-turn lanes).

Technical warrants are an important element of the decision-making process; however, other factors should also be considered when deciding whether to install a left-turn lane, including

- Sight distance relative to the position of the driver.
- Design consistency within the corridor.

These factors should be considered in conjunction with the numerical warrants. For example, if volumes indicate that a left-turn lane is not warranted but there is insufficient sight distance at the location for the left-turning vehicles, then the left-turn lane should be considered along with other potential changes (e.g., remove sight obstructions, realign the highway).

Right-Turn Lane Installation Guidelines

NCHRP Report 279 (*5*) summarizes the then-current (mid-1980s) practice in providing exclusive right-turn lanes (see Table 2-2). The report notes

> No specific warrants or guidelines are apparent for low speed, urban intersections. Engineers generally rely on capacity analyses

Table 2-2. Summary of state design practice in providing right-turn lanes on rural highways (*5*).

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 veh/day	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

Note: DHV = design hourly volume; ADT = average daily traffic; NA = not applicable.

> and accident experience when considering right-turn lanes. In rural areas, focus is primarily on a combination of through and right-turning volume.

Hadi and Thakkar (*10*) investigated the use of through and right-turn movement volumes and speeds as criteria for installing right-turn deceleration lanes. To evaluate the need for right-turn lanes based on these criteria, researchers used as a surrogate the percentage of through vehicles behind right-turning vehicles in the outside (right) lane that performed evasive maneuvers because of the presence of right-turning vehicles. However, this measure could not be estimated from traffic simulation models if these models were used to evaluate the need for right-turn deceleration lanes. They determined that the speed differential between through vehicles affected by right-turning vehicles and those not affected by these turns could be used to determine the need for right-turn deceleration lanes at unsignalized intersections; this was accomplished by using speed differential as a surrogate to safety and related to crash involvement. To determine the total speed differential caused by right-turning vehicles in the outside lane, two variables were needed: the number of through vehicles in the outside lane affected by right-turning vehicles and the average drop in speed of affected vehicles. Their research used these variables to determine critical right-turn volumes that created a speed differential sufficient to necessitate installation of a deceleration lane based on a benefit-cost threshold.

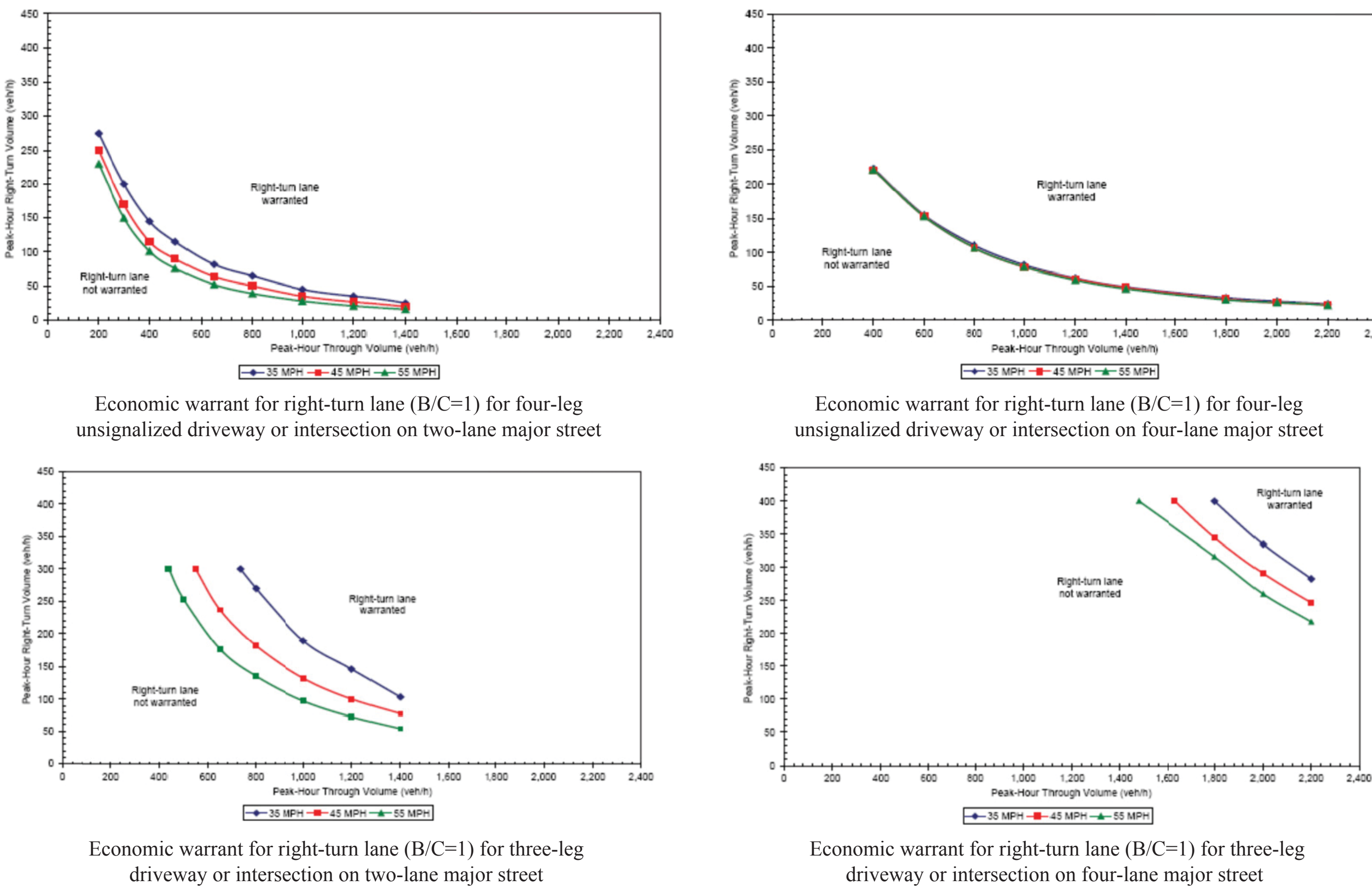
Potts et al. (*11*) used an economic analysis procedure to identify where installation of right-turn lanes at unsignalized intersections and major driveways would be cost effective. The researchers discussed results from their research with respect to right-turn deceleration lanes. They conducted a computer simulation study of motor vehicles and pedestrians at right-turn lanes to determine their operational effects. The researchers determined that right-turn maneuvers from a two-lane arterial at an unsignalized intersection or driveway could delay through traffic by 0 to 6 sec per through vehicle where no right-turn lane was present. Delays to through traffic due to right turns in the same situation on a four-lane arterial were substantially lower, in the range of 0 to 1 sec per through vehicle. They concluded that pedestrians at unsignalized intersections or driveways could have a substantial impact on delay to through vehicles as right-turning vehicles slow to yield to pedestrians, but provision of a right-turn lane could reduce pedestrian-related delays to through traffic by as much as 6 sec per through vehicle, depending on pedestrian volume.

The procedure from NCHRP Project 3-72 (*11*) indicated combinations of through volumes and right-turn volumes for which provision of a right-turn lane would be recommended. Researchers stated that their economic analysis procedure could be applied by highway agencies using site-specific values for ADTs, turning volumes, accident frequency, and construction cost for any specific location (or group of similar locations) of interest. The procedure was used to develop plots that indicated combinations of through volumes and right-turn volumes for which provision of a right-turn lane would be recommended. Examples of such plots are presented in Figure 2-1.

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 Texas *Urban Intersection Design Guide* (*12*) states that the length of left-turn lanes depends on three elements:

- Deceleration length.
- Storage length.
- Entering taper.



Economic warrant for right-turn lane (B/C=1) for four-leg unsignalized driveway or intersection on two-lane major street

Economic warrant for right-turn lane (B/C=1) for four-leg unsignalized driveway or intersection on four-lane major street

Economic warrant for right-turn lane (B/C=1) for three-leg driveway or intersection on two-lane major street

Economic warrant for right-turn lane (B/C=1) for three-leg driveway or intersection on four-lane major street

Source: Potts, I., J. Ringert, D. Harwood, and K. Bauer. Operational and Safety Effects of Right-Turn Deceleration Lanes on Urban and Suburban Arterials. In *Transportation Research Record: Journal of the Transportation Research Board, No. 2023*, Figure 4, p. 60 and Figure 5, p. 61. Reproduced with permission of the Transportation Research Board.

Figure 2-1. Suggested right-turn lane warrants based on results from benefit-cost evaluations (*11*).

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.

Fitzpatrick et al. (*6*) developed recommendations for the approximate total lengths needed for a comfortable deceleration to a stop from the full design speed of the highway. These approximate lengths are shown in Table 2-3 and are based on grades of less than 3%.

On many urban facilities, it is not practical to provide the full length of deceleration for a left-turn lane, and, in many cases, the storage length overrides the deceleration length. In such cases, a part of the deceleration may be accomplished before entering the left-turn lane. A 10-mph differential is commonly considered acceptable on arterial roadways. Higher speed differentials may be acceptable on collector roadways due to higher levels of driver tolerance for vehicles leaving or entering the roadway due to slow speeds or high volumes. Shorter left-turn lane lengths can increase the speed differential between turning vehicles and through traffic. Therefore, the no-speed-reduction lengths given in Table 2-3 should be accepted as a desirable goal and should be provided where practical.

Torbic et al. (*15*) determined that drivers decelerate at rates lower than those identified in the AASHTO Green Book for freeway acceleration lanes, and they often compensate for that by beginning their deceleration in the freeway mainlanes. The result is that drivers do not consistently use the entire length of the speed-change lane for the purpose for which it is provided. It is unknown whether the same principles for deceleration lanes on lower speed freeways may also be applicable to deceleration lanes at intersections.

Storage Length—Single Lane

Long (*16*) reviewed the characteristics of intervehicle spacing for the purpose of auxiliary lane design. He concluded

Table 2-3. Deceleration lengths for left-turn lanes (*6*).

Design Speed (mph)	Deceleration Lengths (ft) from Following Sources:				
	Deceleration Lengths from Other Manuals for Comparison			Deceleration Lengths Determined Using 6.0 ft/sec^2 Deceleration Rate	
	2004 AASHTO Green Book (*4*), page 714	TRB *Access Management Manual* (*13*), page 172	Florida Department of Transportation (FDOT) *2006 FDOT Design Standards* (*14*)	No Speed Reduction in Main Lanes	10-mph Speed Reduction in Main Lanes
30	170	160		170	80
35			145	230	120
40	275	275	155	290	170
45	340		185	370	230
50	410	425	240 (urban) 290 (rural)	460	290
55	485		350	550	370
60		605	405	650	460
65			460	770	550

Note: Blank cells = deceleration length not provided in reference document for the given design speed.

that the value of 25 ft per vehicle used by the CORSIM modeling software was a severe underestimation for determining queue lengths, as was the 3-ft distance between vehicles. He developed new models for estimating average queue lengths and maximum lengths at a given probability; those models were based on an intervehicle spacing of 12 ft, a passenger car length of 15 ft, a 65-ft length for combination trucks, and 30 ft for other vehicles.

Lee, Rouphail, and Hummer (*17*) developed models to predict lane utilization factors for six types of intersections with downstream lane drops and to assess how low lane utilization affects the observed intersection capacity and level of service. They collected traffic and signal data at 47 sites in North Carolina. On the basis of 15 candidate factors, multiple regression models were developed for predicting the lane utilization factor. They compared field-measured delays with delays estimated by the HCM with the use of regression models for lane utilization. They stated that even with the new models for lane utilization, the HCM consistently overestimated delay for all types of lane-drop intersections with low lane utilization and suggested that a reassessment of the effect of lane utilization on capacity may be in order. The study also found that the downstream lane length and traffic intensity positively correlated with the lane utilization factor, existence of a two-way left-turn lane or midblock left-turn bay increased the lane utilization factor, lane drops due to lane usage change had more equal lane volume distribution than the midblock taper lane drop, and some geometric variables at the approach may also influence lane utilization.

Kikuchi et al. (*18*) examined the lengths of turn lanes when a single lane approached a signalized intersection and was divided into three lanes: left-turn, through, and right-turn. Their objective was to determine the appropriate length of each turn lane. From analysis of the vehicle queue pattern at the entrance to the turn lanes, they developed a set of formulas to compute the probabilities of the occurrence of turn-lane overflow and turn-lane blockage. The recommended lane lengths were calculated so that the probabilities that a lane did not overflow and that the entrance of the lane was not blocked were greater than a threshold value of 0.95. Recommended turn-lane lengths, presented in a series of tables, were found to be shorter than those recommended by AASHTO.

In a subsequent study, Kikuchi and Kronprasert (*19*) developed analytical and computational processes for determining the length of the right-turn lane at a signalized intersection. They examined the factors that influenced length, reviewed available literature and practices, derived recommended lengths analytically, and developed a set of tables of recommended lane lengths as a function of approach volumes (right-turn, through-traffic, and cross-traffic volumes) and signal timing. Their analysis compared conditions when right-turn-on-red (RTOR) was not permitted and when it was permitted. Based on achieving desired probabilities of turn-lane overflow and turn-lane blockage, they calculated recommended lane lengths based on the number of vehicle spaces and described a procedure to convert that number to actual distance. They compared their guidelines that account for arrival rates of both right-turn and through vehicles to guidelines that only considered right-turn vehicles; as a result, they concluded their proposed lane lengths were different than those in existing guidelines. Their recommended lengths for RTOR conditions were somewhat shorter than non-RTOR conditions when the right-turn arrival rate was greater than the arrival rate for through vehicles.

Gard (*20*) 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.

Gard compared these equations to the procedures found in the 2000 *Highway Capacity Manual*, the monograph in the Institute of Transportation Engineers' (ITE's) 1988 *Transportation and Land Development*, and the 2-min arrival method in the 2001 AASHTO Green Book. He found that, of the 70 data points for major-street left turns, his method correctly predicted 34% of the observations, and 84% were predicted within one vehicle. In contrast, the Green Book method predicted 35% correctly and 71% 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 (*21*) 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 (*22*) developed suggested storage length values using a procedure 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 on 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 (*3*). 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 (*20*, *21*, *23*).

The draft final report from NCHRP Project 03-91 provides the following discussion regarding storage length for left-turn lanes. 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.

Kikuchi and Kronprasert (*24*) developed an analytical procedure to determine the lengths of left-turn lanes at signalized intersections. They developed a general framework for determining the lengths of the left-turn lanes that prevented lane overflow and blockage of the entrance of the left-turn lane by the queued through vehicles. The framework considered many factors: arrival rates and the sequence of left-turn and through vehicles, different signal schemes, and intersection capacity. Signal schemes included

- Non-exclusive left-turn phasing (Split-Phase).
- Permissive-only (PmO).
- Protected-only leading (PO-Leading).
- Protected-only lagging (PO-Lagging).
- Protected-permissive left-turn phasing (PPLT).

The researchers stated that they identified all possible queue patterns (including the leftover from the previous cycle), and the probabilities of lane blockage and lane overflow were obtained for different combinations of the parameters. Using these parameters, the authors recommended lengths to prevent lane overflow and blockage of more than 95% of the cycles.

Researchers (*24*) noted the effects of several additional variables in their comments associated with the proposed model:

- Percentage of Left-Turn Volume: AASHTO's guides consider only the volume of left-turn vehicles because they are concerned only about the lane overflow. The proposed model suggests the need to consider both through and left-turn volumes to prevent lane blockage as well as lane overflow. For a large percentage of through volume, a length longer than what AASHTO suggests is recommended.
- Protected and Permissive Left-Turn Signal Scheme: The permissive left-turn phase allows left-turning vehicles to clear during the green through phase. As a result, the left-turn lane for the permissive left turn is generally shorter by 10% to 40% than that for the protected left-turn movement.
- Leading and Lagging Left-Turn Signal Schemes: The leading left-turn scheme requires a slightly shorter left-turn lane, 5% to 10%, than the lagging left-turn phase.

- Opposing Traffic for the PPLT Signal Scheme: The volume of the opposing flow has no significant effect on the left-turn lane length as long as the approach left-turn volume is small. When the derived lane length becomes substantially longer than what AASHTO recommends and the space for it is not available, the framework allows the basis to consider changing the signal scheme such that the increase of the length can be kept to a minimum.

According to the Green Book (*4*), 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-min period within the peak hour. Space for at least two passenger cars should be provided; with over 10% truck traffic, provisions should be made for at least one car and one truck. Table 2-4 shows the recommended vehicle length by percent truck included in the TRB *Access Management Manual* (*13*).

Table 2-4. Queue storage length per vehicle (adapted from *13*).

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

Source: *Access Management Manual*, 2003, Table 10-3. Reproduced with permission of the Transportation Research Board.

The 2-min 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* (*8*). The first equation shown in Table 2-5 can be used to determine the design length for left-turn storage as described by the Green Book. For new construction, turning and opposing volumes are typically not mature; accommodation should be made for future growth, and a design year should be chosen that is appropriate for the purpose of the project.

Many states use the Green Book method, a method based on work done by Harmelink (*3*) or a method based on work

Table 2-5. Equations used to determine storage length (*6*).

Equation in TRB *Access Management Manual*

$$L = \frac{V}{N_c} ks$$

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 2-4]).

Equations Used in *NCHRP Report 457*

Equations also used to generate values in Table 2-6

$$P(n > N) = \left(\frac{v}{c}\right)^{(N+1)} \qquad c = \frac{V_o e^{-V_o t_c/3600}}{1 - e^{-V_o t_f/3600}} \qquad N = \frac{ln[P(n>N)]}{ln[v/c]} - 1$$

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

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 2-4]).

by Jack E. Leisch and Associates (*25*) 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* method to determine the expected storage length. The authors of *NCHRP Report 348* (*26*) stated that the required storage length of a left-turn lane depends on the likely left-turn volumes during the peak 15 min 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% of the time and can be estimated by using the cumulative Poisson distribution.

It is generally recognized that a storage area should adequately store the turn demand a large percentage of the time (e.g., 95% or more, which means that the demand would exceed the storage length less than or equal to 5% of the time). A 0.5% limit was used for the major-road left-turn bay lengths in *NCHRP Report 457* based on the recommendation of Harmelink (*3*). 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 2-2 shows 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

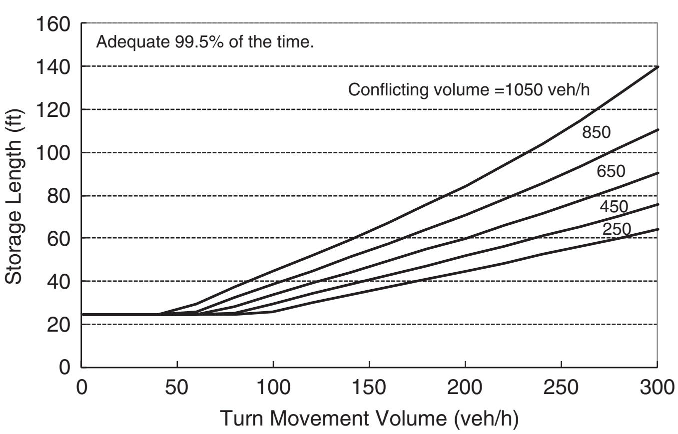


Figure 2-2. Recommended storage lengths for left-turn lanes at uncontrolled approaches using a 25-ft minimum storage length along with a critical gap of 4.1 sec (22).

field studies are used, the storage lengths shown in Table 2-6 are generated.

Each of the sources emphasized 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), the sources 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.

Storage Length—Double Lanes

Kikuchi, Kii, and Chakroborty (*27*) developed a method for estimating the needed length of double left-turn lanes (DLTLs). Their procedure first surveyed how drivers choose a lane of the DLTL in the real world and analyzed the relationship between lane use and the volume of left-turn vehicles. Second, the procedure calculated the probability that all arriving left-turn vehicles during the red phase could enter the left-turn lanes (i.e., no queue spillback of vehicles from the DLTL and no blockage of the DLTL by the queue of through vehicles). This probability was presented as a function of the length of the DLTL and the arrival rates of left-turn and through vehicles. The adequate lane length was derived such that the probability of the vehicles entering the DLTL was greater than a threshold value. Third, the recommended adequate length was expressed in number of vehicles and then converted to the actual distance required based on the vehicle mix and preference between the two lanes. Resulting recommended lengths were presented as a function of left-turn and through volumes for practical application.

The Green Book states that if double left-turn lanes are used, the length required for storage is approximately half that required for single left-turn lanes (*4*).

Taper

Two distinct tapers are commonly defined in many guidelines: approach taper length and bay taper length. An approach taper provides space for a left-turn lane by moving traffic laterally to the right on a street or highway without a median. The bay taper length is a reversing curve along the left edge of the traveled way that directs traffic into the left-turn lane. Illustrations of the use of these tapers along with how the left-turn lane is added to the roadway are shown in the following figures:

- Figure 2-3 shows a left-turn lane added within a median.
- Figure 2-4 shows a left-turn lane added to an undivided two-lane highway where the through lane on the same approach as the added turn lane was shifted to the right

Table 2-6. Recommended storage lengths from *Access Management Manual* equation and *NCHRP Report 457* equations with revised critical gap (*6*).

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

Note: This table assumes 25 ft per vehicle spacing. Table 2-4 provides other suggested spacing lengths based on percent trucks.
[a, b] See Table 2-5 for equations.

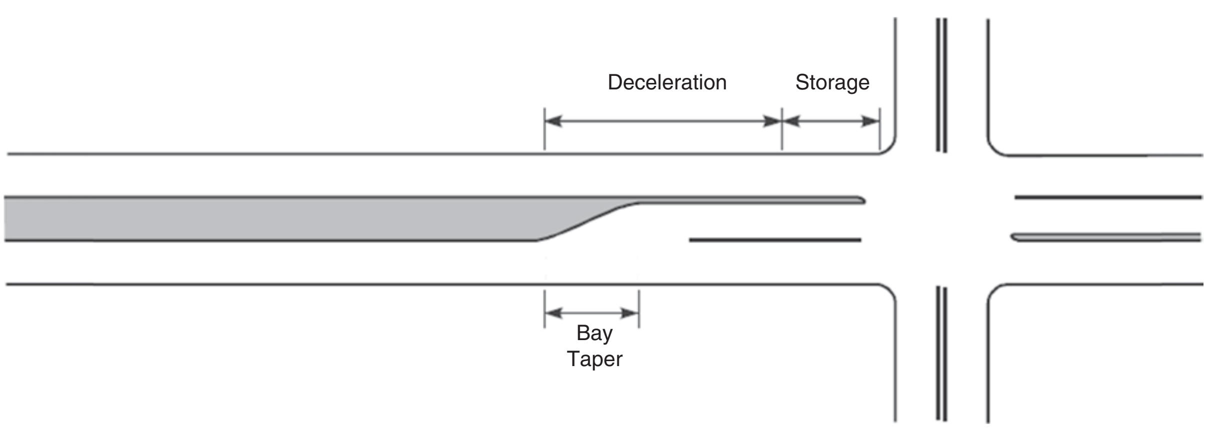


Figure 2-3. Left-turn lane within a median (6).

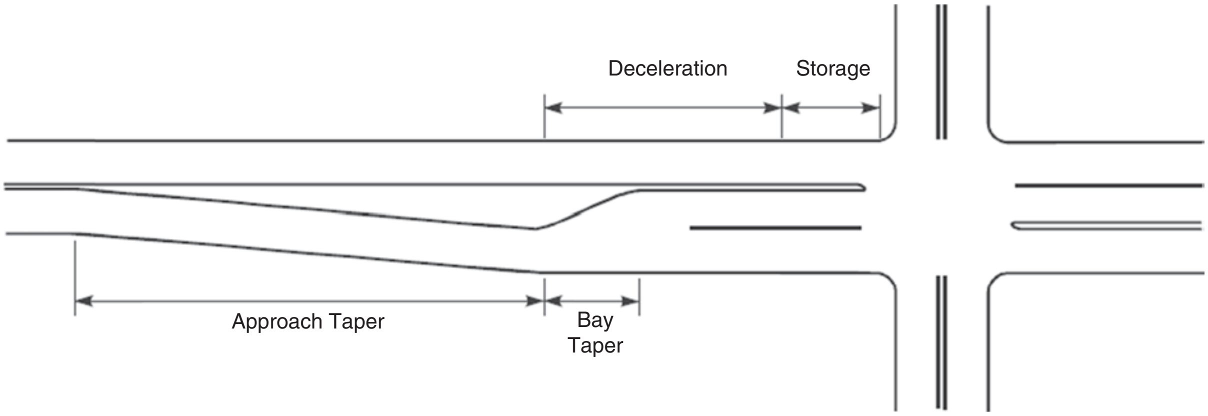


Figure 2-4. Fully shadowed left-turn lane (6).

of the full width of the turn lane. This condition is known as a fully shadowed left-turn lane. Where this configuration is used, it is important that designers follow typical guidelines for lane-addition tapers.

- Figure 2-5 shows a partially shadowed left-turn lane where both through lanes are shifted to provide the needed space for the turn lane. With partially shadowed left-turn lanes, the offset created by the approach taper does not entirely protect or "shadow" the turn lane (*5*).
- Figure 2-6 shows the condition when a lane is added to the outside edge of the approach and the through driver must change lanes to continue traveling straight; otherwise, the driver would be in the left-turn lane. This condition is also known as a bypass lane. Some agencies avoid this layout because of the mixed message to drivers between passing lanes and this condition. For passing or truck climbing lanes, the new added outside lane is for slower-moving traffic, and the inside existing lane is for faster-moving vehicles. For the configuration shown in Figure 2-6, the opposite situation is present; the new added outside lane is for the faster-moving traffic, and the inside existing lane is for the vehicles that are slowing and perhaps stopping while waiting to make the left turn.

Tapers for Left Turns (Bay Taper)

On high-speed highways, it is common practice to use a taper rate between 8:1 and 15:1 (L:T) (*4*). 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 wanting 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 were used. This results in a longer length of full-width pavement for the auxiliary lane. This type of design may reduce

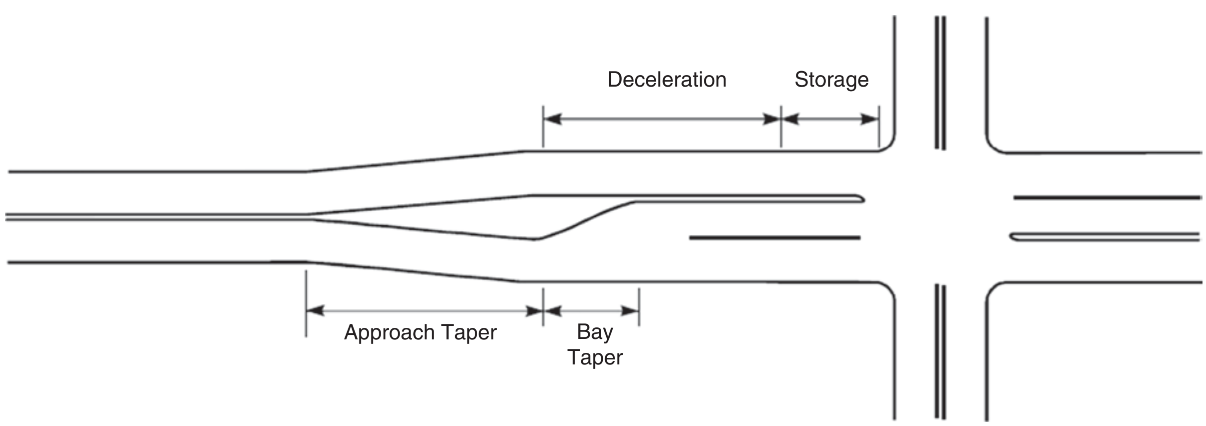
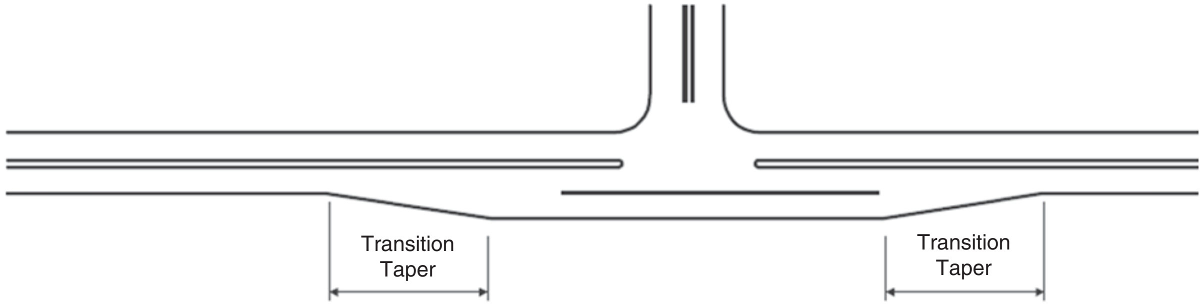


Figure 2-5. Partially shadowed left-turn lane (6).



Note: Some agencies recommend against using this layout because drivers must change lanes to continue traveling straight; otherwise, the driver would be in the left-turn lane.

Figure 2-6. Direct entry into left-turn lane (also known as bypass lane) (6).

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 double-turn lane for urban streets (*4*).

Some agencies permit the tapered section of deceleration left-turn lanes to be constructed in a squared-off or shadowed section at full paving width and depth, particularly where a very short taper is applied. This configuration involves a painted delineation of the taper. The abrupt squared-off beginning of deceleration exits offers improved driver commitment to the exit maneuver and also contributes to driver security because of the elimination of the unused portion of long tapers. The design involves transition of the outer or median shoulders around the squared-off beginning of the deceleration lane.

The Green Book provides advice regarding taper design. The recommended straight-line taper rate is 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 left-turn 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.

Tapers for Through Traffic (Approach Taper)

Though left-turn lanes can be added so that the merge taper guides turning vehicles into the turning lane, certain locations instead use the taper to guide through traffic to the right of the turning lane, as shown in Figure 2-4 and Figure 2-5. Such treatments are often used in rural conditions where it is beneficial to provide added protection and/or guidance to turning vehicles, particularly at isolated T-intersections, where there is no median in which to install a shadowed left-turn lane, and/or where right-of-way is limited. At these locations, through traffic is directed to shift its path, while turning traffic can travel straight into the turning lane. The approach taper is commonly estimated by one of the equations shown in Table 2-7. Table 2-7 also shows comparisons for various speeds and offsets.

While the design guidelines described in the previous section pertain to the use of a bay taper for turning vehicles, similar principles apply when the taper is used to shift the path of through traffic. The taper should be short enough to provide sufficient visual clues to the driver that through traffic must shift, but be long enough to allow the shift to take place at the expected or prevailing operating speed of the roadway. Pavement markings and supplemental signs should be

Table 2-7. Typical length of approach taper to add left-turn lanes (*6*).

Design Speed (mph)	Condition	Equation	Approach Taper Length (ft)	
			6-ft Offset	12-ft Offset
20	Typically used for low-speed approaches (e.g., 40 mph and less)	$L = WS^2/60$	40	80
30			90	180
40			160	320
50	Typically used for high-speed approaches (e.g., greater than 40 mph)	L = WS	300	600
60			360	720
70			420	840

Note: W = width of offset (ft), S = speed (mph).

used to reinforce the action the driver is expected to take. It is important that appropriate vehicle storage length be provided because, with no median protection, the excess queue will extend into the through travel lane. This is one reason why this treatment is more commonly found at isolated intersections with low turning volumes. For similar reasons, it is also important that sufficient deceleration length be provided in the design of the turning lane.

Deceleration and Storage Length for Right-Turn Lanes

The design guide developed as part of NCHRP 3-89 (*28*) notes that speed-change lanes (both for deceleration and acceleration) may be provided to minimize deceleration and acceleration in the through travel lanes. The document also notes:

> There are no generally established criteria concerning where deceleration and acceleration lanes should be provided in conjunction with channelized right-turn lanes. The AASHTO Green Book does not give definitive warrants for the use of speed-change lanes, but identifies several factors that should be considered when deciding whether to implement speed-change lanes: vehicle speeds, traffic volumes, percentage of trucks, capacity, type of highway, service provided, and the arrangement and frequency of intersections.

The NCHRP 3-89 Design Guide (*28*) does not provide guidance regarding right-turn storage length.

Lane or Median Width

FHWA's *Highway Design Handbook for Older Drivers and Pedestrians* (*29*) states that two factors can compromise the ability of older drivers to remain within the boundaries of their assigned lanes during a left turn. One factor is the diminishing ability to share attention (i.e., to assimilate and concurrently process multiple sources of information from the driving environment). The other factor involves the ability to turn the steering wheel sharply enough, given the speed at which they are traveling, to remain within the boundaries of their lanes. Data sources cited by the handbook's authors indicated that a 12-ft lane width provides the most reasonable tradeoff between the need to accommodate older drivers, as well as larger turning vehicles, without penalizing the older pedestrian in terms of exaggerated crossing distance. The handbook's corresponding recommendation is for a minimum receiving lane width of 12 ft, accompanied, wherever practical, by a shoulder of 4 ft minimum width.

Potts, Harwood, and Richard (*30*) investigated the relationship between lane width and safety for roadway segments and intersection approaches on urban and suburban arterials. Their research found no general indication that the use of lanes narrower than 12 ft (3.6 m) on urban and suburban arterials increased crash frequencies. Researchers stated that this finding suggested that geometric design policies should provide substantial flexibility for use of lane widths narrower than 12 ft (3.6 m). They added that inconsistent results suggested increased crash frequencies with narrower lanes in three specific design situations:

- Lane widths of 10 ft (3.0 m) or less on four-lane undivided arterials.
- Lane widths of 9 ft (2.7 m) or less on four-lane divided arterials.
- Lane widths of 10 ft (3.0 m) or less on approaches to four-leg stop-controlled arterial intersections.

The researchers recommended that narrower lanes should be used cautiously in these three situations unless local experience indicates otherwise.

The width of auxiliary lanes should preferably match the width of the through lanes, although they should be at least 10 ft wide (*4*). 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. A suggestion by others, such as (*30*), is that when selecting the width of auxiliary lanes, designers should carefully consider when a narrower lane may not be appropriate in combination with other factors; those factors could include high operating speeds, substantial truck volumes, or restrictive alignments.

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 double 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 (*12*).

As part of NCHRP Project 3-72, Potts et al. (*31*) investigated the relationship of lane width to saturation flow rate on urban and suburban approaches to signalized intersections. Average headways were measured, and then saturation flow rates were calculated at signalized intersection approaches with lane widths of 9.5 to 14 ft. They concluded that saturation flow rate does indeed vary with lane width. Average saturation flow rate was in the range of 1,736 to 1,752 passenger cars (pc)/h/ln for 9.5-ft lanes, 1,815 to 1,830 pc/h/ln for 11- to 12-ft lanes, and 1,898 to 1,913 pc/h/ln for lane widths of 13 ft or greater. These measured saturation flow rates were generally lower than those used in the *Highway Capacity Manual.* Furthermore, the percentage difference in saturation flow rate between sites with 9.5- and 12-ft lanes was about half the value used in the HCM. Because data were limited to queue lengths between 8 and 11 vehicles, the research results did not directly address queue lengths longer than 11 vehicles.

Simultaneous Left Turns

Flexibility in signalization is provided if the left-turn movements are separated (*12*, *32*). This separation, if sufficient, can allow concurrent double left-turn phases. Separate double left-turn phases eliminate the potential problem of overlapping vehicle paths in the intersection.

Safety

Harwood et al. (*33*) conducted a study to investigate the safety effectiveness of left- and right-turn lane treatments. The research team collected geometric design, traffic control, traffic volume, and traffic crash data at 280 improved sites in eight states and at 300 similar intersections not improved during the study period. The types of improvement projects evaluated included installation of added left-turn lanes, installation of added right-turn lanes, installation of left- and right-turn lanes as part of the same project, and extension of the length of existing left- or right-turn lanes. Based on the results of the analyses, they concluded the following:

- Adding left-turn lanes is effective in improving safety at signalized and unsignalized intersections. Installing a single left-turn lane on a major-road approach would be expected to reduce total intersection accidents at rural unsignalized intersections by 28% for four-leg intersections and by 44% for three-leg intersections, with corresponding reductions of 27% and 33% at urban unsignalized intersections. At four-leg urban signalized intersections, installation of a left-turn lane on one approach would be expected to reduce accidents by 10%, and installation on both major-road approaches would be expected to increase, but not quite double, the resulting effectiveness measures for total intersection accidents. Table 2-8 shows a summary of the crash modification factors (CMFs), as presented in Bonneson et al. (*34*).
- Positive results can also be expected for right-turn lanes, with reductions in total intersection accidents of 14% at rural unsignalized locations and 4% at urban signalized locations for installations on single approaches. Installation of right-turn lanes on both major-road approaches to four-leg intersections would be expected to increase, but not quite double, the resulting effectiveness measures for total intersection accidents. Table 2-9 shows a summary of the crash modification factors, as presented in Bonneson et al. (*34*).
- In general, turn-lane improvements at rural intersections resulted in larger percentage reductions in accident frequency than comparable improvements at urban intersections.
- Overall, there was no indication that any type of turn-lane improvement is either more or less effective for different accident severity levels.

In NCHRP Project 17-21, researchers determined state and local agency design practices and policies related to unsignalized median openings for U-turns. After seven categories of midblock and intersection median designs were identified, the research team assessed the designs' effects on safety through field observation and crash data analysis for 115 unsignalized median opening sites with both crash and field data. This knowledge was transferred into design guidelines and a method for comparing the expected safety

Table 2-8. Crash modification factors for adding a left-turn lane at various intersection types (*34*).

Intersection Type	Number of Intersection Legs	Number of Major-Road Approaches with Left-Turn Lanes Installed			
		One Approach		Both Approaches[c]	
		All Crashes	Severe Crashes	All Crashes	Severe Crashes
Rural Signalized	3	0.85	0.86[a]	not applicable	
	4	0.82	0.83[b]	0.67	0.69
Rural Unsignalized	3	0.56	0.45	not applicable	
	4	0.72	0.65	0.52	0.42
Urban Signalized	3	0.93	0.94[d]	not applicable	
	4	0.90	0.91	0.81	0.83
Urban Unsignalized	3	0.67	0.65[e]	not applicable	
	4	0.73	0.71	0.53	0.50

[a] Value estimated as 0.83 = 0.91 / 0.90 × 0.82 using urban intersection data from Harwood et al. (*33*).
[b] Value estimated as 0.86 = 0.91 / 0.90 × 0.85 using urban intersection data from Harwood et al. (*33*).
[c] CMFs for "both approaches" estimated as the square of the "one approach" CMFs.
[d] Data not available from Harwood et al. (*33*). Value estimated using "all crash" data as 0.94 = 0.91 / 0.90 × 0.93.
[e] Data not available from Harwood et al. (*33*). Value estimated using "all crash" data as 0.65 = 0.71 / 0.73 × 0.67.

Table 2-9. Crash modification factors for adding a right-turn lane at various intersection types (*34*).

Intersection Type	Number of Intersection Legs	Number of Major-Road Approaches with Left-Turn Lanes Installed			
		One Approach		Both Approaches[b]	
		All Crashes	Severe Crashes	All Crashes	Severe Crashes
Rural Signalized	3	0.96	0.91[a]	not applicable	
	4	0.96	0.91	0.92	0.83
Rural Unsignalized	3	0.86	0.77	not applicable	
	4	0.86	0.77	0.74	0.59
Urban Signalized	3	0.96	0.91	not applicable	
	4	0.96	0.91	0.92	0.83
Urban Unsignalized[c]	3	0.86	0.77	not applicable	
	4	0.86	0.77	0.53	0.50

[a] Harwood et al. (*33*) did not quantify CMFs for signalized intersections with three legs. They recommended the application of the "four-leg" CMFs to intersections with three legs.
[b] CMFs for "both approaches" estimated as the square of the "one approach" CMFs.
[c] Harwood et al. (*33*) did not quantify CMFs for urban unsignalized intersections. They recommended that the CMFs developed for rural four-leg unsignalized intersections can also be used for urban unsignalized intersections.

performance of different designs to enable engineers in setting policy, establishing project-level design, and discussing the impacts of medians with business and property owners. As documented in *NCHRP Report 524* (*35*), researchers made the following conclusions:

- As medians are used more extensively on arterial highways, with direct left-turn access limited to selected locations, many arterial highways experience fewer midblock left-turn maneuvers and more U-turn maneuvers at unsignalized median openings.
- Field studies at various median openings in urban arterial corridors found estimated U-turn volumes of no more than 3.2% of the major-road traffic volumes at those locations. At rural median openings, U-turn volumes were found to represent, at most, 1.4% of the major-road traffic volumes at those locations.
- Accidents related to U-turn and left-turn maneuvers at unsignalized median openings occurred very infrequently. The 103 median opening study sites on urban arterial corridors experienced an annual U-turn plus left-turn crash average of 0.41. Twelve median openings on rural arterial corridors had an annual average crash total of 0.20. Overall, at these median openings, U-turns represented 58% of the median opening movements and left turns represented 42%. Based on these limited crash frequencies, researchers concluded that there was no indication that U-turns at unsignalized median openings constituted a major safety concern.
- For urban arterial corridors, median opening crash rates were substantially lower for midblock median openings than for median openings at three- and four-leg intersections. For example, the crash rate per million median opening movements (U-turn plus left-turn maneuvers) at a directional midblock median opening was typically only about 14% of the median opening crash rate for a directional median opening at a three-leg intersection.
- Crash rates at directional median openings on urban arterial corridors were lower than at traditional median openings, and conventional three-leg median openings had lower crash rates than corresponding four-leg openings.
- Where directional median openings were considered as alternatives to conventional median openings, two or more directional median openings were usually required to serve the same traffic movements as one conventional median opening. Therefore, researchers concluded that design decisions should consider the relative safety and operational efficiency of all directional median openings in comparison with the single conventional median opening.
- Analysis of field data found that for most types of median openings, most observed traffic conflicts involved major-road through vehicles having to brake for vehicles turning from the median opening onto the major road.
- At urban unsignalized intersections, the research found that installation of a left-turn lane on one approach would be expected to reduce accidents by 27% for four-leg intersections and by 33% for three-leg intersections.
- The minimum spacing between median openings then used by highway agencies ranged from 152 to 805 m (500 to 2,640 ft) in rural areas and 91 to 805 m (300 to 2,640 ft) in urban areas. In most cases, highway agencies used spacings between median openings in the upper end of these ranges, but there was no indication that safety problems resulted from occasional use of median opening spacings as short as 91 to 152 m (300 to 500 ft).

Fitzpatrick, Schneider, and Park (*36*) conducted a study to determine variables that affected the speeds of free-flow turning vehicles in an exclusive right-turn lane and explore the safety experience of different right-turn lane designs. Their evaluations found that the variables affecting the turning speed at an exclusive right-turn lane included the type of channelization present (either lane line or raised island), lane length, and corner radius. Variables that affected the turning speed at an exclusive right-turn lane with island design included (a) radius, lane length, and island size at the beginning of the turn and (b) corner radius, lane length, and turning roadway width near the middle of the turn. The authors compared their study to previous research and found that treatments that had the highest number of crashes were right-turn lanes with raised islands; while in their analysis, they found this type of intersection had the second highest number of crashes of the treatments evaluated in this study as well. In both studies, the "shared through with right lane combination" had the lowest number of crashes. They recommended that these findings be verified through use of a larger, more comprehensive study that includes right-turning volume.

NCHRP Project 16-04 (*37*) was initiated to develop (a) design guidelines for safe and aesthetically pleasing roadside treatments in urban areas and (b) a toolbox of effective roadside treatments to (1) balance the safety and mobility needs of pedestrians, bicyclists, and motorists and (2) accommodate community values. In fulfilling the first of those objectives, researchers discussed auxiliary lanes as an item for consideration. They stated that although many auxiliary lanes have low volumes and may be included as part of a clear zone in the urban environment, higher speed auxiliary lane locations, such as extended length right-turn lanes, are common locations for run-off-road crashes. A lateral offset of 6 ft from the curb face to rigid objects is preferred, and a 4-ft minimum lateral offset should be maintained.

Offset Left-Turn Lanes

The FHWA's *Highway Design Handbook for Older Drivers and Pedestrians* (*29*) recommends that for new or reconstructed facilities, unrestricted sight distance, achieved through positive offset of opposing left-turn lanes, be provided whenever possible. This recommendation is made in anticipation of providing a margin of safety for older drivers who, as a group, do not position themselves within the intersection before initiating a left turn. Where the provision of unrestricted sight distance is not feasible, positive left-turn lane offsets are recommended to achieve the minimum required sight distances appropriate for major roadway design speed and type of opposing vehicle.

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.

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.
- 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 typically required to offset left-turn lanes, but research has shown that offset left-turn lanes can provide significantly greater sight distance for left-turn maneuvers, a particularly critical maneuver for older drivers (*38*). Guidelines were developed for offsetting opposing left-turn lanes at 90-degree intersections on level, tangent sections of divided roadways with 12-ft lanes (*39*). 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 for when both vehicles are unpositioned, and for when 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% 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.

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. (*40*) developed a method for determining the width of the left-turn lane line.

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. 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. 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 (*4*). 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.

Results of a 1996 study by Tarawneh and McCoy (*39*) 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 increased the size of the critical gaps of drivers turning left and also seemed 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 were 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.

While the literature supports the use of offset left-turn lanes, there has been little evaluation of the safety effectiveness of this strategy; a recommendation from the literature is that an investigation is needed to thoroughly evaluate the effectiveness of offset improvements for left-turn lanes in reducing crash frequency and severity at signalized intersections. The safety effectiveness of offset improvements for left-turn lanes is explored empirically in an FHWA Pooled Fund study (*41*, *42*) to provide better support to the states when selecting safety improvements at signalized intersections. Geometric, traffic, and crash data were obtained for 92 installations in Nebraska, 13 in Florida, and 12 in Wisconsin, as well as for some untreated reference sites in each state. To account for potential selection bias and regression-to-the-mean, an Empirical Bayes (EB) before-after analysis was conducted to determine the safety effectiveness of improving the offset for left-turn lanes. Researchers observed a large difference in effects among the three states, which they said may be explained, in part, by the wide variation in offset improvements. Florida and Nebraska employed pavement marking adjustments or minor construction to improve the offset, but while the offset was improved at each site, most improvements did not result in a positive offset. Wisconsin, however, reconfigured left-turn lanes through major construction projects, resulting in substantial positive offsets. Results in Florida and Nebraska showed little or no effect on total crashes, but Wisconsin showed significant reductions in all crash types investigated—total (34%), injury (36%), left-turn (38%), and rear-end (32%).

As part of the Pooled Fund study (*42*), a disaggregate analysis was conducted for Nebraska, the only state with enough installations to disaggregate the results. The analysis revealed that the percent reduction in crashes increased as the expected number of crashes increased. An economic analysis was conducted to identify the level of expected crashes that would yield a crash benefit to justify the construction cost. Based on this analysis, researchers concluded that offset improvement through reconstruction was cost effective at intersections with at least nine expected crashes per year and where left-turn lanes are justified by traffic volume warrants. This study did not address offset improvements at unsignalized intersections. While there are examples of offset improvements at unsignalized intersections in the United States, the authors noted that these results should not be extrapolated to that situation. Rather, they concluded that it is more appropriate to conduct a separate evaluation once there are sufficient installations at unsignalized intersections

Offset Right-Turn Lanes

Many transportation agencies have started using offset right-turn lanes (ORTLs) at two-way stop-controlled intersections in the hope of improving driver safety by providing intersection departure sight distance triangles that eliminate through-roadway right-turning vehicle obstructions. A project in Nebraska (*43*) was initiated to determine when construction of an offset auxiliary right-turn lane is cost effective.

The researchers determined that actual ORTL installations were scarce, leaving little opportunity to develop a robust set of observations. However, they did develop a data collection protocol and collected and analyzed data from a small number of study sites. Results of driver behavior studies at existing locations of offset right-turn lanes indicated that drivers were not performing as expected at parallel-type ORTLs, rendering their presence useless. Taper-type ORTLs appeared to be much more intuitive to driver expectancy and appropriate for the three-dimensional characteristics of all vehicle types. Researchers identified specific negative driver behaviors and recommended appropriate *Manual on Uniform Traffic Control Devices* (MUTCD)-compliant traffic control devices to mitigate misleading visual cues and accentuate elements that reinforce the intended positive behavior at ORTL intersections for successful use of the laterally offset right-turn auxiliary lane. At the completion of their project, researchers made the following conclusions regarding ORTLs:

- There are too few installations to allow safety studies. Ideally, this subject would be a good topic for an NCHRP study because it could use multiple study sites across the nation to collect a large amount of data for a robust statistical analysis.

- There are no geometric guidelines for designers to use when deciding key elements of three-dimensional features of the offset right-turn lane that can generate poor choices by through, right-turning, and left-turning drivers on major roads and stopped drivers at minor road approaches of two-way stop-controlled intersections exhibiting ORTLs.
- Guidelines for typical auxiliary lanes do not appear to be transferrable to ORTLs.
- Optimal guidelines for three-dimensional geometric roadway features evolve over time after the study of behaviors generated by drivers given unfamiliar features in an iterative manner.

Intersection Sight Distance

Yan and Radwan (*44*) 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:

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

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. They also produced similar equations and tables for parallel lanes with offsets and for tapered left-turn lanes.

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 cautioned 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.

Channelization

FHWA's *Highway Design Handbook for Older Drivers and Pedestrians* (*29*) recommends raised channelization with sloping curbed medians rather than channelization accomplished through the use of pavement markings, for the following operating conditions:

- Left- and right-turn lane treatments at intersections on all roadways with operating speeds of less than 40 mph.
- Right-turn treatments on roadways with operating speeds equal to or greater than 40 mph.

Where raised channelization is implemented at intersections, FHWA also recommends that median and island curb sides and curb horizontal surfaces be treated with retroreflectorized markings and be maintained at a minimum luminance contrast level of 2.0 with overhead lighting or 3.0 without overhead lighting.

The design guide developed as part of NCHRP 3-89 (*28*) lists the following reasons for providing a channelized right-turn lane:

- To increase vehicular capacity at intersections.
- To reduce delay to drivers by allowing them to turn at higher speeds.
- To reduce unnecessary stops.
- To clearly define the appropriate path for right-turn maneuvers at skewed intersections or at intersections with high right-turn volumes.
- To improve safety by separating the points at which crossing conflicts and right-turn merge conflicts occur.
- To permit the use of large curb return radii to accommodate turning vehicles, including large trucks, without unnecessarily increasing the intersection pavement area and the pedestrian crossing distance.

A channelized right-turn lane consists of a right-turning roadway at an intersection, separated from the through travel lanes of both adjoining legs of the intersection by a channelizing island. The NCHRP Project 3-89 Design Guide (*28*) states:

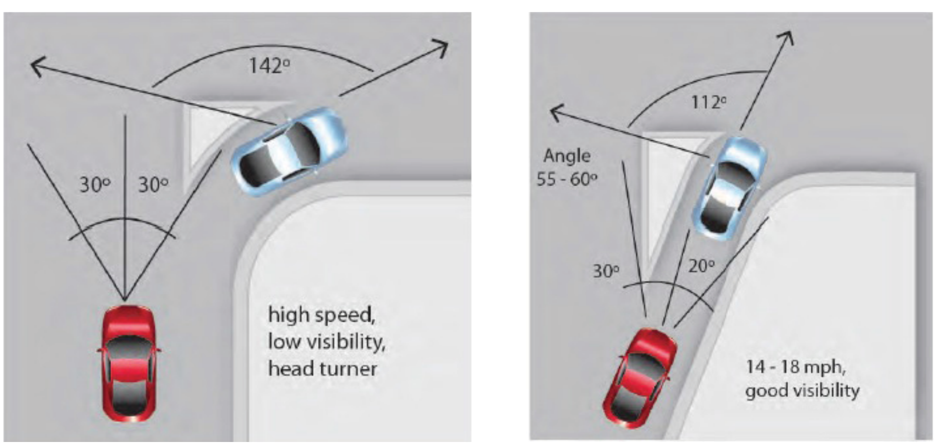
- Curbed islands are considered most favorable for pedestrians because curbs most clearly define the boundary between the traveled way intended for vehicle use and the island intended for pedestrian refuge.

- Orientation and mobility specialists have a strong preference for raised islands with cut-through pedestrian paths because they provide better guidance and information about the location of the island for pedestrians with vision impairment than painted islands.
- When right-turn volumes are high and pedestrian and bicycle volumes are relatively low, capacity considerations may dictate the use of larger radii, which enable higher-speed, higher-volume turns. Increasing the radius of a channelized right-turn roadway reduces right-turn delay by approximately 10 to 20% for each 5-mph increase in turning speed.
- Small corner radii, which promote low-speed right turns, are appropriate where such turns regularly conflict with pedestrians, as higher speeds have been shown to result in a decrease in yielding to pedestrians by motorists. The alignment of a channelized right-turn lane and the angle between the channelized right-turn roadway and the cross street can be designed in two ways (as illustrated in Figure 2-7):
  - A flat-angle entry to the cross street (island shaped like an equilateral triangle, often with one curved side). This design is appropriate for use in channelized right-turn lanes with either yield control or no control, such as locations with an acceleration lane, for vehicles at the entry to the cross street.
  - A nearly right-angle entry to the cross street (island shaped like an isosceles triangle). The nearly right-angle entry design can be used with Stop sign control or traffic signal control for vehicles at the entry to the cross street; yield control can also be used with this design where the angle of entry and sight distance along the cross street are appropriate.

Effect of Skew

Skew angles are an important design element for any approach to an intersection, but for certain auxiliary lanes (e.g., uncontrolled right-turn lanes) they are even more of a factor. Son, Lee, and Kim (*45*) developed a method for calculating sight distance available to drivers at skewed unsignalized intersections. The method considered that the sight distance may vary depending on (1) driving positions of the drivers and (2) different lines of sight given to drivers by different types of vehicles with unique sight-line obstructions. They derived equations and nomographs for calculating available sight distance that included the influence of factors such as geometry (e.g., intersection angle, lane width, shoulder width, and position of stop line), vehicle dimensions, and the driver's field of view. They concluded that findings from their research provided evidence that a skew angle greater than 20 degrees should not be used in design when the design vehicle is a large vehicle or semitrailer because available sight distances were less than the required stopping sight distance, even with a low value of design speed for intersection angles less than 70 degrees.

The *Highway Design Handbook for Older Drivers and Pedestrians* (*29*) recommends establishing 15 degrees as a minimum skew angle as a practice to accommodate age-related performance deficits at intersections where right-of-way is restricted; at skewed intersections where the approach leg to the left intersects the driver's approach leg at an angle of less than 75 degrees, the prohibition of RTOR is recommended. The handbook cites multiple studies documenting restricted neck movement in older drivers, making detection of and judgments about potential conflicting vehicles on crossing



Channelized Right-Turn Lane With Flat-Angle Entry to Cross

Channelized Right-Turn Lane With Nearly Right-Angle Entry to Cross Street

Source: FHWA PEDSAFE, 2013 Guide, found at: http://www.pedbikesafe.org/PEDSAFE/

Figure 2-7. Typical channelized right-turn lane with differing entry angles to the cross street (adapted from 46).

roadways much more difficult. The *Highway Safety Manual* (*1*) includes crash modification factors for skew angle. A 2013 study (*47*) developed crash modification factors for three-leg and four-leg intersections and concluded that the minimum critical angle for intersections in roadway design policies should be revised to 75 degrees.

Multiple Turn Lanes

Several projects have documented experiences with multiple turn lanes over the last 25 years, but a comprehensive study on optimal design features is not available. This section will summarize some of the experiences documented in the literature.

Wortman (*48*) conducted a state-of-the-art review on double left-turn lanes for the Arizona Department of Transportation in the 1980s. In it, he cited work from Neuman (*5*) that suggested double left-turn lanes should be considered at any signalized intersection with high left-turn design-hour demand volumes and a general rule-of-thumb of 300 veh/hr or more as the appropriate demand volume for consideration of the double left-turn lanes. Other guidelines from Neuman include the following:

- The throat width for the turning traffic is the most important design element. Drivers are most comfortable with extra space between the turning queues of traffic. Because of the offtracking characteristics of vehicles and the relative difficulty for acceptance of two abreast turns, a 36-ft throat width is desirable for acceptance of two lanes of turning traffic. In constrained situations, 30-ft throat widths are acceptable minimums.
- Guiding pavement markings to separate the turning lanes are recommended. The then-current MUTCD recommended 2-ft long dashed lines with 4-ft gaps to channelize turning traffic. These channelization lines should be carefully laid out to reflect offtracking and driving characteristics.

Wortman also cited some studies that showed that the capacity of either lane in a double left-turn lane approach was lower than a single left-turn lane (e.g., a lane in a double left-turn might have 80% of the capacity of a single left-turn lane) and that the inside (median-side) left-turn lane had a somewhat lower capacity than the outside (shoulder-side) left-turn lane.

Brich (*49*) examined positive guidance pavement markings for double left-turn lanes in Virginia. In that study, he cited another finding from Neuman (*5*) that double left-turn lanes operate at approximately 1.8 times the capacity of a single left-turn lane.

Hurley (*50*) developed a mathematical model, based on field data, to predict auxiliary lane use for an intersection that uses double left-turn lanes onto a freeway entrance ramp, where the right lane is dropped in advance of the merge area. He compared this condition with downstream lane reduction to the typical intersection configuration in the *Highway Capacity Manual* and found that the collected data indicated that lane utilization factors were considerably larger than the default value presented in the HCM. He added that this was not unexpected since the *Highway Capacity Manual* does not take downstream lane reduction into consideration. However, for intersections with long turn and auxiliary lane lengths and high flow rates, he expected that at least some of the data would fall somewhat close to the HCM default value. This did not happen, even though outside left-turn lane lengths as long as 119 m, downstream auxiliary lane lengths as long as 463 m, and total left-turn flow rates as high as 1,288 veh/hr were encountered at the intersections studied.

NCHRP Report 505 (*51*) discusses characteristics of intersection geometry to accommodate trucks. The authors referenced the existing Green Book guideline that the desirable turning radius for a double left-turn lane is 90 ft, but they concluded that there was little else on the design of multiple turn lanes to account for turning trucks. They stated that the primary factor to consider in designing double left-turn lanes is vehicle offtracking or swept path width. When vehicles negotiate the turn side by side, the vehicles should not encroach on the adjacent travel lane. Because many factors affect the control turning radius of double left-turn lanes, they stated that it is necessary to provide guidance on the range of offtracking or swept path width of design vehicles for various turning radii. They determined offtracking and resultant swept path widths of several design vehicles for 90-degree turns with centerline turning radii of 50, 75, 100, and 150 ft using AutoTURN software. Based on their analysis, they recommended that an exhibit be included in the Green Book that indicates the swept path width of several design vehicles for centerline turning radii of 75, 100, and 150 ft, to provide flexibility in designing adequate turning paths for double left-turn lanes by allowing for interpolation of swept path widths for a range of turning radii.

Cooner et al. (*52*) conducted research in Texas on design and operations of triple left-turn (TLT) and double right-turn (DRT) lanes. Previously existing guidelines were nearly nonexistent, so the research was conducted to develop guidelines for elements such as geometric design, appropriate signs and markings, and signal timing. The field studies in Texas collected both static (e.g., lane widths, grades, pavement markings, traffic signs, upstream and downstream conditions, and signal timings) and dynamic (e.g., volumes by lane, saturation flow, and critical events) data in order to evaluate design and operational performance. Researchers collected the data at five TLT and 20 DRT lane sites, primarily in the Dallas–Fort

Worth and Houston urban areas. They reported the following as key findings for TLT lanes:

- Lane utilization patterns were varied for each of the five sites studied.
- All sites were T-intersections with peak-hour volumes from 646 to 2,846 vehicles.
- Lighted pavement markers used to delineate the lane lines between the TLT lanes were effective at reducing violations and well received by the public at one site.
- Saturation flow rates in Texas were consistent with earlier published national values.

They also reported these important operational findings for DRT lanes:

- Most vehicles used the outside lane (closest to the curb) to make their right turns.
- Peak-hour volumes ranged from a low of 200 to a high of almost 1,000 vehicles.
- Lane utilization (inside vs. outside) was comparable when the right-turn volumes were high.
- Saturation flow rates were higher in the inside lane (average = 1,717 veh/hr versus the outside lane at 1,668 veh/hr) and also generally lower than those at TLT sites.
- The impact of trucks in the inside lane was greater than when in the outside lane.

A review of crash diagrams along with a field conflict study and comparison study led the authors to conclude that the TLT lanes they studied did not experience any major safety issues. They also concluded that, in general, a well-designed DRT lane does not cause significantly higher crash frequency or severity compared to single right-turn lanes.

Some of the key recommendations based on the research include

- TLT lanes should be considered when turning volumes exceed 600 veh/hr.
- DRT lanes should be considered when turning volumes exceed 300 veh/hr.
- Clear turning guide lines (i.e., dotted lines marking the turning path) are highly recommended for both sides of the inside right-turn lane when the intersection has a turning angle greater than 90 degrees.
- Narrow DRT lanes (i.e., turning roadway width less than 30 ft) with channelization should not be used.
- RTOR is not advised for the inside lane when there are more than two receiving lanes.
- Designers should avoid installing DRT lanes near access points (e.g., corner gas stations).
- If an auxiliary receiving/acceleration lane is provided for the curb right-turn lane at channelized double turn lanes, its length should not be less than 150 ft.
- For closely spaced intersections, if a downstream intersection uses DRT lanes, the outside (curb) lane should not be aligned with any through lane at the upstream intersection.

Researchers added that TLT and DRT lanes are not appropriate for all situations and an operational analysis should support their use. They suggested that other techniques (e.g., grade separation, signal timing) might be better solutions for a particular site, especially when considering the effects of adjacent intersections, pedestrian/bicycle movements, and other key factors.

Bypass Lanes

NCHRP Project 3-91 (*6*) examined existing guidelines for installation of bypass lanes, also called blister lanes, at intersections on two-lane rural highways. Bypass lanes (see Figure 2-8) are provided to relieve congestion due to left-turning vehicles in the travel lane. This alignment requires the through driver to change lanes to continue through the intersection; it may be used where right-of-way is constrained but a left-turn lane is warranted. Researchers developed suggested left-turn treatment installation warrants based on results from benefit-cost evaluations for rural two-lane highways. Warrant information based on a benefit-cost ratio of 1.0 as a threshold is available in Fitzpatrick et al. (*6*).

Though the bay taper length is not needed on a bypass lane, it is still necessary to provide the necessary approach taper to

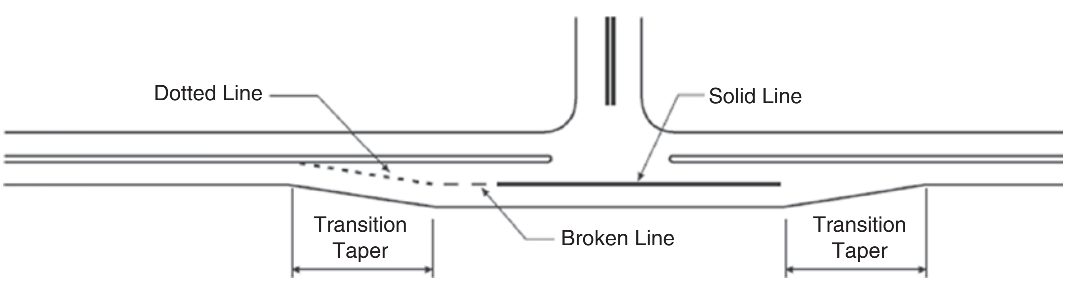


Figure 2-8. Example of bypass lane with markings (6).

guide through traffic around the left-turn lane. The typical length of the approach taper for a bypass lane is the same as that used for a shadowed or partially shadowed lane when through traffic is shifted to the right. Depending on the configuration of the intersection, it is also necessary for a bypass lane to have a departure taper as the through traffic is brought back into its original alignment. For the example shown in Figure 2-8, the departure taper for eastbound through traffic would be equal to the approach taper necessary to add the left-turn lane for the westbound turning traffic. The departure taper associated with the intersection in Figure 2-6 is illustrated in Figure 2-8.

A similar departure taper would be appropriate at a T-intersection where a left-turn lane is added in only one direction. In the case of Figure 2-8, if there is no south leg of the intersection, and an eastbound left-turn lane is installed, the same departure taper would bring the through traffic back into the two-lane alignment east of the intersection.

Bypass lanes are more commonly used at T-intersections than at intersections with four legs because traffic is being diverted in only one direction with no cross traffic on the stem of the T. Use of bypass lanes at four-leg intersections must incorporate proper guidance to approaching drivers so that they do not continue through the turning lane into oncoming traffic. In addition, appropriate sight distance must be provided so that through drivers in the bypass lane have a clear view of through and left-turning cross traffic, and vice versa.

Regardless of the number of legs at the intersection, it is often useful to provide additional guidance to through drivers that they need to change lanes at bypass lanes. This can be accomplished through broken pavement markings or dotted lines that have a much shorter stroke length and shorter spacing than standard markings that permit passing. This marking is commonly called a "skip-stripe" and is illustrated in Figure 2-8. This marking reinforces the message that through drivers must change lanes but still permits left-turn drivers to travel straight into the turning lane.

Passing Lanes

An increasingly common treatment on rural two-lane highways is the addition of periodic passing lanes on corridors that may not be ready for expansion to a full four-lane alignment. These corridors are known by different names in different locations, but they are generally defined as two-lane rural highways in which periodic passing lanes have been added to allow passing of slower vehicles and the dispersal of traffic platoons. Passing lanes are typically provided in both directions of travel, either alternating from one direction of travel to the other or operating side by side within a section of roadway, allowing passing opportunities in both directions.

In some parts of the United States, particularly in Texas, such corridors are known as "Super 2" corridors, while in Europe they are commonly known as "2+1" corridors. Super 2 projects can be introduced on an existing two-lane roadway where there is a significant amount of slow-moving traffic, there is limited sight distance for passing, and/or the existing traffic volume has increased, thus creating the need for vehicles to pass on a more frequent basis.

Although Super 2 corridors have demonstrated benefits in reducing delay and improving opportunities to pass, their operational performance at intersections is not as well known. A recent Texas project by Brewer et al. (*53*) examined Super 2 operations for corridors with higher volumes (ADT of 5,000 to 15,000 veh/day). Driveways were not a primary focus of the project, but researchers conferred with Texas Department of Transportation (TxDOT) engineers to determine common practices, compared with findings from field studies and simulation. Researchers ultimately made only a general recommendation regarding the location of driveways, stating that where practical, designers should avoid substantial traffic generators such as state highways or high-volume county roads or driveways within passing lanes, or consider providing an auxiliary lane (for left turns or right turns, as applicable) if the traffic generator falls within the limits of a Super 2 passing lane.

An NCHRP-sponsored scan tour looked at characteristics of 2+1 roads in several European countries to determine the potential applications of the design for use in the United States. The NCHRP authors (*54*) concluded that the benefits of 2+1 roads in Europe validated a recommendation for their use in the United States, to serve as an intermediate treatment between an alignment with periodic passing lanes and a full four-lane alignment. They also recommended that 2+1 roads were most suitable for level and rolling terrain, with installations to be considered on roadways with traffic flow rates of no more than 1,200 veh/hr in a single direction. The authors recommended that major intersections should be in the buffer or transition areas between opposing passing lanes, with the center lane used as a turning lane.

Alternative Intersection Designs

Some 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 (*55*) 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 (*56*) 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 (e.g., urban, suburban, and rural) and previous forms of traffic control (e.g., two-way stop and signal). Statistical analysis revealed a 35% 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. Multilane roundabouts have accessibility requirements that single-lane roundabouts do not have. The 2011 *Proposed Guidelines for Pedestrian Facilities in the Public Right-of-Way* (*57*), submitted by the United States Access Board in its Notice for Proposed Rulemaking, contains a requirement that an accessible pedestrian-activated signal (e.g., a traditional traffic control signal or a pedestrian hybrid beacon) should be provided for each multilane segment of each pedestrian street crossing, including the splitter island. Such signals are also required to clearly identify which pedestrian street crossing segment the signal serves. The accessibility requirements will have an effect on operations as well as construction and maintenance costs.

Rodegerdts et al. (*56*) further concluded that drivers at roundabouts in the United States appear 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.

Acceleration Lanes

To increase the capacity of through traffic at signalized intersections, additional lanes with limited length—called auxiliary lanes—are added to the roadway on the approach and departure of the intersection. These are often added for vehicles turning right off the arterial at the intersection, though they may also be used by through vehicles and by vehicles turning right and accelerating onto the arterial. Because of their limited length, as well as other factors, these lanes are not as fully used as other continuous through lanes. Tarawneh (*58*) conducted a research project to (1) observe and identify the level of use of auxiliary through lanes added at intersections of four-lane, two-way roadways and (2) study the effects of auxiliary lane length, right-turn volume, and through/right-turn lane group delay on the level of their use. He collected lane use data during 1,050 saturated cycles at eight signalized intersections with different auxiliary lane lengths, and all factors investigated—auxiliary lane length, right-turn volume, and stopped delay—were found to contribute significantly to the use of auxiliary lanes at the 0.01 level. The level of each factor's contribution, however, was dependent on the level of the other two. Longer auxiliary lanes, lower right-turn volumes, and excessive approach delays encouraged the use of auxiliary lanes by straight-through vehicles, and observed lane use by through vehicles ranged between one and seven vehicles per cycle. The range of lane utilization adjustment factors (f_{LU}-factors) calculated from field data was 0.73 to 0.82, which was lower than the then-current 1997 *Highway Capacity Manual* (*59*) default value of 0.91 for a three-lane through/right-turn group.

Other studies have reviewed characteristics of acceleration lanes and speed-change lanes, but those studies typically focus on freeway interchanges instead of at-grade intersections. One study in particular, by Torbic et al. (*15*), revealed that drivers neither accelerate at constant rates, nor at rates as high as those identified in the AASHTO Green Book. In addition, drivers in free-flow conditions do not typically use the entire length of the speed-change lane when it is provided. It is not clear whether the same principles for lower speed acceleration lanes on freeways may also be applicable to acceleration lanes for intersections.

The design guide developed as part of NCHRP Project 3-89 (*28*) notes:

> Acceleration lanes provide an opportunity for vehicles to complete the right-turn maneuver unimpeded and then accelerate parallel to the cross-street traffic prior to merging. The addition of an acceleration lane at the downstream end of a channelized right-turn lane can reduce the right-turn delay by 65 to 85%, depending on the conflicting through traffic volume, and may be considered where right-turn delay is a particular problem. Channelized right-turn lanes with acceleration lanes appear to be very difficult for pedestrians with vision impairment to cross. Therefore, the use of acceleration lanes at the downstream end of a channelized right-turn lane should generally be reserved for locations where no pedestrian or very few pedestrians are present. Typically, these would be locations without sidewalks or pedestrian crossings; at such locations, the reduction in vehicle delay resulting from addition of an acceleration lane becomes very desirable.

Design Tools

Kindler et al. (*60*) described the development of an expert system for diagnostic review of at-grade intersections on rural two-lane highways. This system, the Intersection Diagnostic Review Module (IDRM), was developed as a component of the Interactive Highway Safety Design Model (IHSDM) to aid designers in assessing the safety consequences of geometric design decisions, particularly for combinations of geometric features. IDRM was developed to allow such problems to be identified and evaluated in an automated and organized fashion. IDRM identifies concerns by using models of the criticality of specific geometric design situations. These include existing geometric design models—like sight distance models—as well as newly developed models. IDRM uses 21 specific models to address 15 high-priority issues related to the intersection as a whole and to individual approach legs. IDRM makes no attempt to select a particular treatment as appropriate to the intersection. After further investigation, the IDRM user may select a particular treatment as appropriate on the basis of the available evidence and engineering judgment, or the user may conclude that no treatment is necessary and that the project should be built as designed. Thorough documentation of the development of IDRM is found in a separate FHWA report (*61*). The report focuses on documenting the knowledge base developed for the IDRM software. It also documents the software in that it identifies the knowledge structure, problem definitions, models, decision algorithms, formulas, and parameter values implemented in the software.

Schurr et al. (*62*) developed a model to describe design speed profiles of vehicles traversing horizontal curves on approaches to stop-controlled intersections on two-lane two-way rural highways. They used the model to create a procedure for designing horizontal curves that would accommodate vehicles transitioning from high speeds to a stop. Based on speed profile data from 15 study sites in Nebraska, the researchers concluded that posted speed, median type, presence of rumble bars, roadway surface condition, and degree of rutting did not significantly affect the vehicle speed profiles at these sites at a 95% confidence level. They also concluded that the intercepts of the regression lines for approaches with and without horizontal curves were significantly different in the case of heavy vehicles. The speed of heavy vehicles on tangent approaches was generally about 8 mph higher than on sites that exhibited horizontal curvature, though the rate of deceleration remained almost the same until vehicles were near the stop. Passenger cars exhibited no statistically significant difference between curved and tangent alignments. Researchers used the results of the study to develop a procedure for determining the minimum curve radius appropriate for a roadway alignment approaching a stop ensuring that (1) the visual expectations of the driver were met, (2) the comfort of the passengers within the vehicle was optimized, (3) the curve design used a simple curve with no spirals, (4) the vehicle speed within the limits of the curve was reasonable, (5) sufficient braking distance to the stop was available, and (6) deceleration rates were reasonable. Although this project did not specifically focus on auxiliary lane design, the guidelines related to deceleration profiles of vehicles on stop-controlled approaches could be relevant to vehicles approaching a turning lane.

Ongoing and Recently Completed Research

Researchers reviewed some sources for additional research projects not complete during Task 2. A summary of selected research projects is provided in this section.

Auxiliary through lanes beyond signalized intersections are recognized as an approach to increase the intersection capacity through their efficient use. The benefits from the use of auxiliary through lanes at signalized intersections can be realized in the presence of equally distributed traffic over the lanes prior to the intersection. Use of auxiliary through lanes beyond a signalized intersection has been seen throughout the United States. Prior studies suggest that the length of the auxiliary lane beyond the intersection is a significant factor affecting the upstream lane usage and therefore the intersection capacity. However, the conditions for their effective use and their effect on safety, operation, and the environment have yet to be documented. Thus, research is needed to provide technical assessment of their use, document their effect on safety and operations, and develop guidelines including design criteria and placement. A recent NCHRP project was tasked with assisting traffic engineers in adopting criteria for the effective and safe use of intersection auxiliary through lanes. The objectives of the research were to provide guidelines and procedures to analyze, justify, and design auxiliary through lanes at signalized intersections. The results of the research report were published in 2011 as *NCHRP Report 707* (*63*) and *NCHRP Web-Only Document 178* (*64*).

Many transportation agencies use channelized right-turn lanes to improve operations at intersections, although their effect on safety for motorists, pedestrians, and bicyclists is not clear. The Americans with Disabilities Act (ADA) requires that all pedestrian facilities, including sidewalks and crosswalks, be accessible to pedestrians with disabilities. The U.S. Access Board has published draft rights-of-way guidelines requiring pedestrian signals at channelized turn lanes. Research in *NCHRP Report 674* (*65*) addressed this issue. Regardless of the outcome of that research, some agencies may remove existing channelized right-turn lanes and avoid constructing new ones. Guidance is needed to help make these decisions based on reliable data on their safety impacts. NCHRP Project 3-89 (*66*) was initiated to develop design guidance for channelized right-turn lanes, based on balancing the needs of passenger cars, trucks, buses, pedestrians (including pedestrians with disabilities), and bicycles. For this project, a channelized right-turn lane is characterized by separation from the through and left-turn lanes on the approach by an island and separate traffic control from the primary intersection. The channelized right-turn lane may or may not have a deceleration lane entering it, and it may have a merge or an auxiliary lane at the exiting end. The revised final report has been received and will be published as an NCHRP report.

A current NCHRP project (*67*) is examining existing access management policies and guidelines. These policies often include elements such as guidelines for auxiliary lanes and turning lanes at intersections. The objective of this study is to develop research-based guidelines for access management. This research will culminate in model access management and design guidelines and procedures for various roadway classifications and design types. It will also address how these criteria might vary in the context of different roadside environments. The resulting guidelines will be accompanied by detailed rationale on their benefits and application so they may be readily adapted and applied by state transportation agencies and local governments or metropolitan planning organizations (MPOs) through their transportation planning and design processes.

Review of Online Design Manuals

As part of Task 2 efforts, the research team conducted a state-of-the-practice review of current design considerations. A review of state online design manuals was conducted to identify (1) what is being discussed at the state level and (2) current design criteria being used for intersection auxiliary lanes. The review focused on the following design elements and policy components:

- Queue storage length.
- Entering taper length.
- Deceleration length.
- Turn-lane width.
- Channelization/island design.
- Offset left-turn lanes.
- Double (or dual) left-turn lanes.
- Bypass lane for left turns.
- Right-turn lanes.
- Advice/warrants on when to install.
- Pedestrians.
- Free-flow or channelized right turn.
- Reference to Green Book in policy.

Researchers found 42 state manuals on line, 36 of which contained information on one or more of the items in the above list. Table 2-10 summarizes the guidance provided in the state manuals.

Table 2-10. Summary of auxiliary lane design guidelines.

	Queue Storage Length	Entering Taper Length	Deceleration Length	Turn-Lane Width	Channelization/ Island Design	Offset Left-turn Lanes	Double (or Dual) Left-Turn Lanes	Bypass Lane (Left Turn)	Right-Turn Lanes	When to Install Advice	Pedestrians	Free Right Turn	Reference Green Book
Green Book	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	
Alaska	✓	✓	✓						✓			✓	✓
Arizona	✓	✓	✓	✓		✓	✓				✓	✓	
California	✓	✓	✓	✓	✓	✓	✓		✓	✓	✓	✓	
Colorado	✓	✓	✓	✓			✓		✓	✓		✓	✓
Connecticut	✓	✓	✓	✓		✓	✓	✓	✓	✓			
Delaware	✓	✓	✓	✓					✓			✓	✓
Florida	✓	✓	✓	✓					✓		✓		✓
Georgia	✓	✓	✓	✓	✓			✓					✓
Idaho													✓
Illinois	✓	✓	✓	✓	✓	✓	✓		✓	✓	✓	✓	
Indiana	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓		
Iowa	✓		✓	✓						✓			
Kansas	✓	✓	✓						✓				
Kentucky	✓	✓	✓	✓					✓	✓	✓		
Louisiana	✓	✓			✓					✓			✓
Maine	✓	✓	✓	✓			✓	✓		✓			
Massachusetts	✓	✓	✓						✓		✓		✓
Minnesota	✓	✓	✓	✓	✓		✓	✓			✓	✓	✓
Mississippi	✓	✓	✓							✓			
Missouri	✓	✓	✓	✓		✓			✓	✓			✓
Montana	✓	✓	✓	✓	✓	✓	✓		✓	✓		✓	
Nebraska	✓	✓	✓	✓	✓				✓	✓		✓	✓
Nevada		✓		✓						✓			✓
New Jersey	✓	✓	✓	✓									
New York	✓	✓	✓		✓	✓	✓		✓	✓		✓	✓
North Carolina	✓	✓	✓		✓	✓			✓	✓			✓
Ohio	✓	✓	✓	✓								✓	
Oregon										✓			
Pennsylvania	✓	✓	✓	✓		✓							✓
South Dakota	✓	✓	✓	✓						✓			
Tennessee	✓	✓	✓	✓						✓			
Texas	✓	✓	✓	✓			✓		✓		✓		
Utah	✓	✓	✓	✓						✓			✓
Virginia	✓	✓	✓	✓	✓		✓		✓	✓	✓		✓
Washington	✓	✓	✓	✓	✓				✓	✓	✓		
Wisconsin	✓	✓	✓	✓	✓	✓				✓	✓		

Note: No online manual found for Alabama, Arkansas, Hawaii, Maryland, New Mexico, Rhode Island, South Carolina, Wyoming. Online manual found, but with no information on auxiliary lanes, for Michigan, New Hampshire, North Dakota, Oklahoma, Vermont, West Virginia.

CHAPTER 3

State of the Practice

Introduction

As part of Task 2 efforts on NCHRP Project 3-102, the research team conducted a state-of-the-practice review of current design considerations. The focus of this sub-task was to identify those practices and evaluations not adequately documented in traditional literature. To accomplish that objective, the research team contacted representatives from a selection of state DOTs to inquire about current practices and potential guidance needs, based on their professional experience and the policies of their respective departments. The information presented in this chapter reflects the participants' responses as provided to the research team.

Based on the findings from the Task 1 review of published literature and of state design manuals and guidance documents, the research team developed a preliminary list of topics for further study. Researchers then enlisted the help of the project panel, whose feedback guided the refinement of that list into a shorter list of three main topics: the design of deceleration lanes, the design of multiple turn lanes, and the design of right-turn islands. These three topics were the focus of the state-of-the-practice review.

Researchers conducted this review through the use of a written questionnaire, distributed to the aforementioned DOT personnel. Researchers developed the questionnaire with additional review and feedback by the project panel and one additional DOT representative who did not formally complete the questionnaire but offered suggestions for improvements to the document. Using the finalized questionnaire, researchers contacted representatives from DOTs in 12 states; these states were selected because they satisfied one or more of the following criteria:

- The DOT's existing design guidance identified in Task 1 contained information on the design of multiple turning lanes.
- The DOT's existing design guidance identified in Task 1 contained information on right-turn island design.
- The state was a participant in Highway Safety Information System (HSIS).
- The state was known by researchers and/or panel members to have experience with one or more of the focus topics.

The selected states were California, Colorado, Connecticut, Florida, Illinois, Maine, Maryland, Minnesota, North Carolina, Ohio, Texas, and Washington.

Researchers used the AASHTO Subcommittee on Design and Geometric Design Technical Committee member rosters to identify the initial list of representatives to contact in each of the selected states; this was supplemented by a search of DOT websites to find specific contact information. For California and North Carolina, researchers contacted members of the project panel instead of AASHTO subcommittee members. Researchers contacted all representatives via e-mail and included the questionnaire document as an attachment. In the e-mail, representatives were asked to respond initially to verify their willingness to participate and to state whether they preferred to complete the questionnaire by e-mail or by telephone interview.

In many cases, the contacted representatives either forwarded the questionnaire to others in their office or enlisted the help of others to complete the questionnaire as a group. Indications from respondents were that staff members from Traffic Engineering or other divisions were commonly involved with auxiliary lane installations along with those who worked on roadway geometric design.

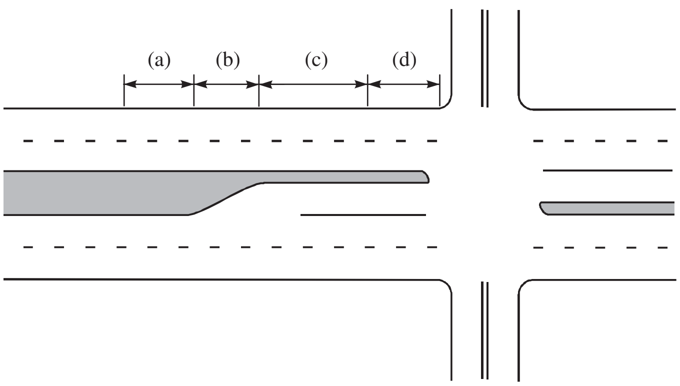
Completed questionnaires were received from 11 of the 12 states contacted; no response was received from Connecticut. For each responding state, the completed questionnaire was returned via e-mail, although two respondents also asked to be contacted by phone to discuss the questions and ensure that their understanding was consistent with researchers' expectations.

The remainder of this chapter contains the responses to the questionnaires, with summary comments to note trends and patterns in the respondents' answers. The summary tables and associated text presented in the remainder of this chapter are restatements of the responses provided by the survey participants to indicate a current practice, as intended by the design of the survey. The research team makes no statement about the appropriateness of any of the policies or practices mentioned.

Questionnaire Responses

Deceleration Length

D-1. Using the figure below, how do you define a "deceleration lane" for a left- or right-turn lane at an at-grade intersection? Please note all of the dimensions that apply to your definition.



This question was asked to determine whether the respondents were considering the same definition of "deceleration lane." While deceleration can take place in any of the dimensions shown in the figure, the dimension (c) is the formal design element included for deceleration of turning vehicles for purposes of this research. Dimension (a) is part of the through travel lane, (b) is the lane-addition taper, and (d) is the queue storage length.

Perhaps verifying the prudence of including this question, researchers received various responses. All of the responses included the dimension (c), but only 3 of the 11 respondents indicated that (c) was their definition of a deceleration lane. Two states formally used the term "deceleration length" or "deceleration distance" instead; they defined that length as the sum of (b) and (c). Other states included both the taper and the full-width portion of the lane (i.e., b plus c) as their deceleration distance. Table 3-1 summarizes all answers to this question.

Table 3-1. Summary of responses to Question D-1.

Answer	Frequency (n=11)
(c)	3
(b)+(c)+(d)	3
(b)+(c)	4
(c)+(d)	1

D-2. Briefly summarize your understanding of your agency's policy or guidance for the design of a deceleration lane.

Respondents' answers typically focused on the criteria for determining whether a lane should be installed and/or how to determine the length. For the latter category, the speed of approaching vehicles was most frequently mentioned, listed either directly or implicitly by nine respondents. This includes three specific mentions of the AASHTO Green Book, which uses design speed as the criterion for deceleration length. Other respondents mentioned entry speed, design speed, or posted speed. One respondent also specifically stated that "accepted practice is for some deceleration (10 mph) to be assumed to occur in the through lane and to consider the taper as part of the deceleration lane."

To determine both the need for a deceleration lane and its appropriate length (for deceleration and/or for storage), a capacity or demand analysis was mentioned by five respondents. Two also included consideration of restricted geometrics or crash issues as potential installation criteria. One respondent added discussion of appropriate widths for medians, left-turn lanes, and right-turn lanes.

Table 3-2 summarizes the responses received. Some respondents provided more than one criterion, so sums may exceed the number of overall responses (n = 11).

Table 3-2. Summary of responses to Question D-2.

Criteria for Determining Installation and/or Length	Frequency (n=11)
AASHTO Green Book (2001 or 2004)	5
Capacity	3
Entry speed	3
Restricted geometrics	2
Crash issues	2
Design speed	2
Posted speed	1
Arrival demand	1
Storage demand	1

Table 3-3. Summary of responses to Question D-3.

Answer	Frequency (n=11)
(e)	2
(a)	1
(a)+(b)	1
(a)+(d)	1
(a)+(e)	1
(b)+(e)	1
(a)+(b)+(e)	2
(b)+(c)+(d)+(e)	1
all	1

D-3. On which of the following is your agency's policy based? (Please note all that apply.)

a. AASHTO Green Book.
b. *NCHRP Report 279* (*Intersection Channelization Design Guide*).
c. Harmelink's procedures.
d. TRB *Access Management Manual.*
e. Other (please specify).

Respondents were free to list as many sources as needed, and there were multiple combinations of responses. Altogether, the Green Book was cited by seven respondents as at least one source they use, the most common of the specific source choices provided, followed closely by *NCHRP Report 279* (five times). However, the "other" category was the most frequently cited (eight times), and some agencies use more than one "other" source in their policy. Table 3-3 lists the responses to this question.

"Other" sources cited include

- AASHTO *Guide for the Development of Bicycle Facilities.*
- AASHTO *Roadside Design Guide.*
- California *Highway Design Manual.*
- Colorado *State Highway Access Code Manual.*
- FHWA *Manual on Uniform Traffic Control Devices for Streets and Highways.*
- Florida DOT *Project Traffic Forecasting Handbook.*
- Florida DOT *Plans Preparation Manual.*
- Florida DOT *Intersection Design Guide.*
- Florida DOT report *Triple Left Turn Lanes at Signalized Intersections.*
- Ohio *Location and Design Manual.*
- TRB Highway Capacity Manual.

D-4. If constraints prevent you from installing the preferred length of deceleration lane, how do you decide which elements are reduced? Is your decision based on a quantitative or qualitative analysis?

The typical response to this question was that the decision is a qualitative one based on the conditions at the site. Respondents indicated that many factors (e.g., available right-of-way, distance between adjacent intersections, and signal timing) could affect how the available deceleration lane length is allotted. Four respondents stated that they would initially assume that deceleration occurred in the through lane and would correspondingly reduce the deceleration length of the auxiliary lane, while one stated that the taper length would be shortened. One other respondent stated that the agency's minimum turning lane length is 100 ft but did not specify how that might be divided among the various design elements.

As the previous answers might suggest, this decision is frequently qualitative rather than quantitative; six respondents directly answered that this was the case. Responses to this question are summarized in Table 3-4.

D-5. Is there guidance on designing deceleration lanes not currently included in your state policy or in the AASHTO Green Book *that would be useful? If so, what?*

Respondents were generally satisfied with the guidance contained in their state manuals and in the AASHTO Green Book. Two respondents offered suggestions for additional material, one of which provided suggestions on various topics already contained within the state's guidance documents. Seven participants responded that current guidance is adequate, and one provided no answer to the question. Responses are summarized in Table 3-5.

Table 3-4. Summary of responses to Question D-4.

Element(s) Reduced	Frequency (n=11)
Depends on existing site conditions	5
Deceleration length (i.e., deceleration occurs in through lane)	4
Taper	1
Minimum turn lane length is 100 ft	1
Nature of Decision	**Frequency (n=11)**
Qualitative	6
Quantitative	1
Could be either/Varies	1
Unknown	1
No answer	2

Table 3-5. Summary of responses to Question D-5.

Additional Guidance Suggested	Frequency (n=11)
How to develop projected turning movements (and corresponding storage lengths)	1
Type of widening (e.g., symmetrical, pocket, and near-side) for left-turn treatments, left-turn warrants, left overs, and super streets (for Green Book)	1
Findings from state research projects are added to the guidance if applicable	1
None (i.e., current guidance is adequate)	7
No answer	1

D-6. Please list up to three locations in your state with installations that you would consider "best practice" sites.

Respondents from five states provided a total of 10 locations or corridors for consideration. The sites are listed in Table 3-6.

D-7. Do you have sites that are less than optimal? If so, what improvements do you consider needed and why?

Six respondents indicated that there were sites they would like to improve; most spoke generally, though one respondent provided locations of some specific examples. The descriptions of the sites in each state were similar: turning lanes were not long enough and/or they were not wide enough. Lack of available right-of-way was specifically mentioned as a causal factor by two respondents. Lengthening the turning lane to improve capacity was a desired improvement listed by five respondents, while one would like to have wider lanes in some locations. Also, one respondent listed sight distance for opposing left-turn lanes as a common problem that could be improved through offsetting the turning lanes.

Multiple Turn Lanes

Double Left-Turn Lanes

2L-1. Does your agency's policy include guidelines for when it is appropriate/recommended to install a second left-turn lane?

Only 1 of the 11 respondents indicated that his/her agency's policy did not have specific guidelines. Two of the remaining ten respondents provided some details in addition to their affirmative response. One stated that the recommendation to construct multiple turn lanes is based on the results of traffic studies performed during project development and later refined during preliminary design. The other stated that the decision was largely specific to the location or the demand, and that all double left-turn lanes are signalized with protected phasing; considerations included left-turn demand (especially above 300 vehicles per hour [veh/hr]), capacity, storage, and safety. This respondent also indicated that sometimes intersections have an ultimate geometry that includes a double left-turn lane but short-term forecasts have volumes that are not yet high enough to justify more than one lane, which requires intermediate design solutions.

2L-2. If so, briefly summarize your understanding of your agency's policy or guidance for the design of double left-turn lanes.

The 11 respondents typically discussed the criteria that should be met to install a double left-turn lane, and those criteria were usually a turning volume that exceeded 300 veh/hr and/or the inability to provide a single left-turn lane that had sufficient length or level of service (LOS) to meet demand. Several respondents specifically mentioned that an analysis of operational characteristics (usually through microsimulation) was conducted to determine both current and long-term needs. Three respondents specified that double left-turn lanes always operated under fully protected signalization.

Table 3-6. Summary of responses to Question D-6.

State	City/County	Site
Colorado	(not specified)	SH 391 (MP 0.97)/SH 8
Colorado	(not specified)	SH 121 (MP 6.53)/W. Quincy Ave
Florida	West Palm Beach	State Road 7 @ Old Hammock Way
Florida	Tallahassee	Capital Circle SE @ Tram Road
Florida	Tampa	State Road 597 @ Stall Road
Maine	Winthrop	Route 202 and Main Street
Maine	Winthrop	Route 202 and Highland Avenue
Minnesota	(not specified)	"Various turn lanes along Trunk Highway 371 between Little Falls and Baxter which incorporated 500-ft long bay lengths"
North Carolina	Fuquay Varina/ Wake County	W. Academy St. at Coley Farm Rd
North Carolina	Cary	Westbound SR 1010 (Ten Ten Road) at Kildaire Farm Road

Table 3-7. Summary of responses to Question 2L-2.

Installation Criteria/Design Feature	Frequency (n=11)
Consider when turning volume exceeds 300 veh/hr	6
Not possible or practical to provide single lane of sufficient length or LOS will not be met	3
Conduct evaluation of operational characteristics	3
Fully protected (green arrow) signal phasing	3
Design elements similar to single left, but with additional clearance distance to accommodate second lane	1
Consider for turning volume of 200-600 veh/hr	1
Major signalized intersections where high peak-hour left-turn volumes are expected	1

Table 3-7 summarizes the responses to this question; respondents often had more than one component to their answers, and all responses are listed, so the summation is greater than the number of respondents.

2L-3. Does your agency's policy discuss the design of receiving lanes on the departure of the double left-turn lane? If so, briefly summarize your understanding of your agency's policy.

Most of the survey respondents (8 of 11) indicated that their policy contained some specific guidance for receiving lanes. Typical values were 30 ft minimum and 36 ft desired width for the turning path or turning curve, which would also be the total width of the receiving lanes. Three respondents mentioned that when designing for trucks (e.g., WB-40 or larger), their turning curve was designed for an SU design vehicle on the inside lane and a larger design vehicle on the outside lane, rather than simultaneous turning of two large vehicles. Two respondents discussed the transition from the full width of the turning curve down to the normal lane width; one stated that the turning width was tapered down to the normal width at a ratio of 15:1 to 20:1, while the other stated that the full-width receiving lane should extend for a minimum of 1,500 ft. Table 3-8 summarizes the responses to this question.

2L-4. What is the estimated capacity of a typical double left-turn lane?

a. Twice the capacity of a single left-turn lane
b. 1.8 times the capacity of a single left-turn lane
c. 1.5 times the capacity of a single left-turn lane
d. Other (please specify)

Different values have been proposed for this variable, so the research team wanted to determine what was typical based on the practices and experiences of the participating agencies. The most common answers were (b) and (d), with three responses each. Within (d), the respondents said that they determined the value site by site with a traffic or operational study and/or the HCM methodology. Two respondents answered (c), and one respondent answered (a). Responses are summarized in Table 3-9.

2L-5. Compared to a typical single left-turn lane, what adjustments do you make to queue storage length, deceleration length, and/or taper length for a double left-turn lane?

Respondents provided various answers to this question. Answers categorized by the three design elements are summarized in Table 3-10.

2L-6. Is there guidance on designing double left-turn lanes not currently included in your state policy or in the AASHTO Green Book *that would be useful? If so, what?*

Respondents were typically satisfied with the guidance contained in their state manuals and in the AASHTO Green Book. Five respondents offered suggestions for additional material, one of which provided suggestions on various topics already contained within the state's guidance documents. Five participants responded that current guidance is adequate,

Table 3-8. Summary of responses to Question 2L-3.

Typical Turning Curve Width	Frequency (n=11)
36 ft (desirable); 30 ft (minimum)	4
Two receiving lanes of 15 ft each	1
35 ft	1
26-45 ft, depending on radius and percentage of trucks	1
"Full width" receiving lanes	1
No width specified	3

Table 3-9. Summary of responses to Question 2L-4.

Answer	Frequency (n=11)
(b)	3
(d) with traffic study/HCM	3
(c)	2
(a)	1
No answer	2

and one provided no answer to the question. Responses are summarized in Table 3-11.

2L-7. Please list up to three locations in your state with double left-turn installations that you would consider "best practice" sites.

Respondents from six states provided a total of 14 locations or corridors for consideration. The sites are listed in Table 3-12.

2L-8. Do you have double left-turn sites that are less than optimal? If so, what improvements do you consider needed and why?

Eight respondents provided suggestions for improvements; most spoke generally, although one respondent provided locations of some specific examples. The descriptions of the improvements in each state were similar: longer taper lengths, longer storage lengths, wider lanes, and better guidance to prevent offtracking.

Triple Left-Turn Lanes

3L-1. Does your agency's policy include guidelines for when it is appropriate/recommended to install a third left-turn lane?

Only two respondents indicated that their state's policy included guidance for triple left-turn lanes, though two others

Table 3-10. Summary of responses to Question 2L-5.

Storage Length	Frequency (n=11)
Site-by-site/traffic analysis	3
According to HCM	2
0.6 times single lane	1
Enough to avoid through vehicles blocking entrance	1
No guidance available	1
No answer	3
Deceleration Length	**Frequency (n=11)**
Sum of storage and taper	2
Based on 15:1 taper (4:1 minimum)	1
Site-by-site/traffic analysis	1
No guidance available	1
No answer	6
Taper Length	**Frequency (n=11)**
Twice single lane	2
Same taper rate as single lane	1
Site-by-site/traffic analysis	1
15:1 taper (4:1 minimum)	1
Extended by 50 ft	1
No guidance available	1
No answer	4

Table 3-11. Summary of responses to Question 2L-6.

Additional Guidance Suggested	Frequency (n=11)
Add a design chart that shows the left-turn lengths for two left-turn lanes, along with taper lengths, as a function of design speeds (for Green Book)	1
Guidance on designing double left-turn lanes will be further developed in future publications of the state manual and will be presented in the agency's intersection design class	1
For access control, use a barrier curbed median adjacent to the left-turn lanes and the approaching median taper	1
Directional Left Over and Super Street typical designs included in state Roadway Design Manual (for Green Book)	1
Different sets of bay taper lengths for urban and non-urban areas, and procedure to select the proper signal phasing sequence for intersections with left-turn lane overflow and/or blockage problems (based on state research project)	1
None (i.e., current guidance is adequate)	5
No answer	1

Table 3-12. Summary of responses to Question 2L-7.

State	City/County	Site
Colorado	(not specified)	SH 391 (MP 0.97)/SH 8
Colorado	(not specified)	SH 391 (MP 4.23)/W. Alameda Ave
Colorado	(not specified)	SH 95 (MP 12.09)/W. 72nd Ave
Florida	West Palm Beach	State Road 7 @ State Road 80
Florida	Tallahassee	Capital Circle SE @ Blair Stone Road
Florida	Tampa	State Road 580 @ Waters Avenue
Maine	Augusta	Route 202 (Western Ave) and Armory Street
Maine	Augusta	Whitten Road at Route 202
Maine	Portland	Route 1 at the Route 703 Connector
Minnesota	Roseville	SB Trunk Highway 51 at County Road B
North Carolina	Leland/Brunswick County	US-17 SuperStreet Corridor
North Carolina	Cary	Tryon Road at Kildaire Farm Road
North Carolina	Chapel Hill/Orange County	US 15-501 and Erwin Road
Washington	Olympia	Black Lake Blvd for both the left-turn channelization SB for EB or SB 101 ramp

suggested that there might be conditions under which a site-specific analysis of conditions would produce a triple left-turn as an alternative. The remaining respondents did not have such guidance and, in fact, often suggested that such treatments were discouraged or prohibited, typically because of truck traffic.

3L-2. If so, briefly summarize your understanding of your agency's policy or guidance for the design of triple left-turn lanes.

The two affirmative responders in Question 3L-1 stated that they both referred to guidance by the Florida DOT. The two conditional responders both stated that they reviewed intersections on a site-by-site basis.

3L-3. Does your agency's policy discuss the design of receiving lanes on the departure of the triple left-turn lane? If so, briefly summarize your understanding of your agency's policy.

One affirmative respondent provided these criteria:

- Need to design so the relative turning movement distribution at a downstream intersection does not compromise the ability of the receiving lanes to store the left-turning vehicles.
- Three downstream lanes need to be available to receive the left-turning traffic for at least 300 ft from the intersection, and at least two continuous downstream lanes must exist beyond that point. If possible, continuous downstream receiving lanes should be provided to avoid a lane drop.
- The receiving leg should have a raised median island at least 2 ft wide to provide drivers on the inside lane with a visual point of reference to guide the vehicle through the left-turn maneuver.

The other affirmative respondent stated that they specify 15-ft lanes at the throat, dotted lines for guidance through the turn, and a minimum width of 15 ft at the center of the curve.

One conditional respondent stated that his/her agency's goal in conducting microsimulation analysis was to optimize the amount of width on the receiving lanes to accommodate the design vehicle in each turn lane individually but not simultaneously. The other site-specific response emphasized lanekeeping and lane continuity issues and concerns.

3L-4. Compared to a typical single left-turn lane, what adjustments do you make to queue storage length, deceleration length, and/or taper length for a triple left-turn lane?

One affirmative respondent stated that the taper length would simply be the additional length created by the same taper rate used for the other lanes and all other design elements would be determined after analysis of site-specific conditions (e.g., geometry, right-of-way, and traffic). The other three respondents offered similar comments.

3L-5. Is there guidance on designing triple left-turn lanes not currently included in your state policy or in the AASHTO Green Book *that would be useful? If so, what?*

The two affirmative respondents suggested material from Florida. One recommended a 2002 FDOT report entitled "Triple Left Turn Lanes at Signalized Intersections," while the other suggested that existing FDOT guidelines would be further developed in future editions of that state's policies and design classes. One respondent with no current agency policy advised that inclusion of basic parameters and other control criteria would be helpful to designers, and another referred to a DOT-sponsored research project.

Table 3-13. Summary of responses to Question 3L-6.

State	City/County	Site
Colorado	(not specified)	SH 128 (MP 7.97)
Colorado	(not specified)	E. Orchard Road @ SH 83 (MP 66.56)
Colorado	(not specified)	S. Chambers Road @ SH 83 (MP 66.92)
Florida	Lake Worth	State Road 7 @ State Road 802
Florida	Wellington	State Road 7 @ State Road 882
Florida	West Palm Beach	State Road 7 @ State Road 704
Minnesota	Minneapolis	The southbound exit ramp from I-494 to Trunk Highway 62—southbound to eastbound turning movement
Washington	(not specified)	Southbound Interstate 5 and Eastbound State Route 512

3L-6. Please list up to three locations in your state with triple left-turn installations that you would consider "best practice" sites.

Respondents from four states provided a total of eight locations or corridors for consideration. The sites are listed in Table 3-13.

3L-7. Do you have triple left-turn sites that are less than optimal? If so, what improvements do you consider needed and why?

One state's respondent indicated that there may be less-than-optimal sites in operation; needed improvements might include additional storage due to increases in traffic volumes that have exceeded expectations. One state offered specific examples of sites that could be improved.

Double Right-Turn Lanes

2R-1. Does your agency's policy include guidelines for when it is appropriate/recommended to install a second right-turn lane?

Four of eleven respondents stated that they had guidance for installation of double right-turn lanes; two of them also indicated that the guidelines were similar to those for double left-turn lanes.

2R-2. If so, briefly summarize your understanding of your agency's policy or guidance for the design of double right-turn lanes.

Nine participants provided responses to this question, indicating that, even if they do not have specific guidance in their agencies' policies, relevant guidelines in place can be applied if needed. Table 3-14 summarizes the responses to this question.

2R-3. Does your agency's policy discuss the design of receiving lanes on the departure of the double right-turn lane? If so, briefly summarize your understanding of your agency's policy.

Five respondents stated that their policies discussed receiving lanes; of those five, three stated that the guidance was similar to that for double left-turn lanes. One stated simply that free right turns were discouraged, and one said that the design was somewhat contradictory because it challenged the designer to accommodate large trucks while still trying to minimize radii for intersection and clearance widths.

2R-4. Compared to a typical single right-turn lane, what adjustments do you make to queue storage length, deceleration length, and/or taper length for a double right-turn lane?

While double right-turn lanes were not that common among the respondents, almost all of them offered a response as to

Table 3-14. Summary of responses to Question 2R-2.

Policy Highlights	Frequency (n=11)
Site-by-site analysis	3
Length is the same as double left-turn lanes	1
Mirror image of double left-turn lanes	1
Similar to double left-turn lanes; consider priority of pedestrian movements, use channelizing islands, and discourage right-turn on red	1
Double right-turns are discouraged, but they can be considered when a single turning lane does not have adequate length, storage, or level of service	1
Increase turning radius to 200 ft	1
Rarely used, but would require larger radius and throat width	1
No answer/Not applicable	2

Table 3-15. Summary of responses to Question 2R-4.

Adjustments	Frequency (n=11)
Similar to double left-turn lanes	4
Site-by-site analysis	2
Half the storage length	1
Per HCM	1
No guidance available	1
Not applicable/No answer	2

appropriate adjustments to make. Answers are summarized in Table 3-15.

2R-5. *Is there guidance on designing double right-turn lanes not currently included in your state policy or in the AASHTO* Green Book *that would be useful? If so, what?*

Five respondents offered suggestions for additional guidance, as shown in Table 3-16.

2R-6. *Please list up to three locations in your state with double right-turn installations that you would consider "best practice" sites.*

Respondents from four states provided a total of seven locations or corridors for consideration. The sites are listed in Table 3-17.

2R-7. *Do you have double right-turn sites that are less than optimal? If so, what improvements do you consider needed and why?*

Five agencies responded to this question; the common thread was the need for improved storage, along with possible deceleration length, taper length, and/or pedestrian accommodation. One agency described two specific sites, while others did not know of exact locations.

Island Design for Right-Turn Lanes

I-1. *Does your agency's policy include guidelines for when it is appropriate/recommended to install a channelizing island at a right-turn lane?*

Ten of the eleven respondents stated that they have available guidance on channelizing islands in one or more of their policy/guidance documents. The remaining respondent indicated that his/her agency has some guidance in an Access Permit Manual, although it was primarily used by developers and not agency designers.

I-2. *If so, briefly summarize your understanding of your agency's policy or guidance for the design of a channelizing island adjacent to a right-turn lane.*

Table 3-16. Summary of responses to Question 2R-5.

Additional Guidance Suggested	Frequency (n=11)
General guidance, with emphasis on pedestrian considerations	1
Design chart that shows the right-turn lengths for two right-turn lanes, along with taper lengths, as a function of design speeds; develop and illustrate warrants for double right-turn lanes	1
Describe conditions and solutions for when the capacity of a double right-turn is exceeded	1
Additional design considerations that must be addressed for successful operation when changing from single to double turning lane	1
Findings from DOT-sponsored research project "Development of Guidelines for Triple Left and Dual Right Turn Lanes"	1
None (i.e., current guidance is adequate)	5
No answer	1

Table 3-17. Summary of responses to Question 3L-6.

State	City/County	Site
Colorado	(not specified)	SH 83 (MP 65.31)/E. Arapahoe Rd
Florida	Wellington	State Road 7 @ State Road 882
Florida	West Palm Beach	State Road 7 @ State Road 704
Florida	Tallahassee	State Road 20 @ State Road 373
Maine	Auburn	Veterans Bridge Connector at Route 4
Maine	Topsham	295 NB off ramp at Route 196
Washington	Olympia	Double-right from ramp terminal for WB or NB 101 to Black Lake Blvd

Respondents provided a wide variety of answers, reflective of the open-ended nature of the question. Responses are summarized below:

- Traffic islands can be beneficial in separating lanes of traffic, directing a traffic stream toward a specific direction, or providing pedestrian refuge areas. Traffic islands can also be used to discourage/prohibit certain movements.
- Islands are included in intersection design for the following purposes:
  - Separating conflicts.
  - Control of angle of conflict.
  - Reducing excessive pavement areas.
  - Regulating traffic and indicating proper use of intersection.
  - Arrangements to favor a predominant turning movement.
  - Protecting pedestrians.
  - Protecting and storing turning and crossing vehicles.
  - Locating traffic control devices.
  - Access control.

  The size of the islands should be a minimum of 50 sq ft in urban areas, and 75 sq ft in rural areas; 100 sq ft minimum preferred in all areas. Triangular islands should be a minimum of 12 ft (preferably 15 ft) on a side after rounding of corners. Islands shall be delineated, depending on size. Design Manual references triangular island layouts in Chapter 9 (Intersections) (pages 634–639) of the 2004 Green Book.
- Policy is based on geometrics. At larger intersections where large turning radii are required due to vehicle mix, channelizing islands may be appropriate. In locations where mast arms may be required due to proximity to the coast, channelizing may be used to site mast arm poles.
- Provide a smooth, free-flowing alignment both into and out of the channelization.
- Turning roadways are channelized areas (separated by an island) which allow a moderate-speed, free-flowing right turns away from the intersection area. The designer should consider using turning roadways when
  1. It is desirable to allow right turns at speeds of 15 mph or more;
  2. The angle of turn is greater than 90°;
  3. The volume of right turns is high, the turning movement is from a high-volume road, or it is desirable to remove right turns away from a signal;
  4. It is desirable to reduce the intersection paved area. As a guide, if an island with a turning roadway will be at least 75 square ft (urban) or 100 square ft (rural), then a turning roadway should be considered; and/or
  5. Pedestrian volumes are high and a pedestrian refuge is a desirable feature.
- There are two conditions for using a channelized right-turn lane. One would be a free right-turning movement and the other is when we are trying to protect the right-turning movement from last-minute right-turn lane encroachment at an intersection which would negatively affect the through movement in an intersection.
- They may be warranted in the following situations:
  1. When it is necessary to accommodate semitrailer or large buses.
  2. When intersections are skewed.
  3. When it is desirable to allow right turns at speeds of 15 mph or more with an acceleration lane.
  4. When it is necessary to accommodate traffic control signal installation.
- Complete Street efforts under development and improved curb ramp designs are addressing/providing improved right-turn island designs. Most locations where this will be pursued will be retrofit intersections or locations with new development improvements with pedestrian, modal, and bicycle access. Traditional approach was mobility-based and aimed at free flow or yield optimal conditions; the evolving guidance we are using is more modal-sensitive and improves vehicular alignment for a lower speed but hopefully safety approach and turn in a multi-modal environment.
- Channelizing islands control and direct traffic into the proper paths for the intended use and are an important part of intersection design. They may be of many shapes and sizes, depending on the conditions and dimensions of the intersection. A common form is the corner triangular shape that separates right-turning traffic from through traffic. Channelizing islands are used at intersections for the following reasons:
  - Separation of conflicts.
  - Control of angle of conflict.
  - Reduction in excessive pavement areas.
  - Indication of proper use of intersection.
  - Favoring of a predominant turning movement.
  - Pedestrian protection.
  - Protection and storage of vehicles.
  - Location of traffic control devices.

  These islands should be placed so that the proper course of travel is immediately obvious and easy for the driver to follow. Care should be given to the design when the island is on or beyond a crest of a vertical curve, or where there is a substantial horizontal curvature on the approach to or through the channelized area. Properly placed islands are advantageous where through and turning movements are heavy.

- Where the inner edges of pavement for right turns at intersections are designed to accommodate semitrailer combinations or where the design permits passenger vehicles to turn at 15 mph or more (i.e., 50 ft or more radius), the pavement area at the intersection may become excessively large for proper control of traffic. In these cases, channelizing islands should be used to more effectively control, direct, and/or divide traffic paths. Physically, islands should be at least 50 ft^2 in urban and 75 ft^2 for rural conditions (100 ft^2 preferable for both) in size and may range from a painted to a curbed area.
- Traffic islands perform the following functions:
  – Channelization islands control and direct traffic movements.
  – Divisional islands separate traffic movements.
  – Refuge islands provide refuge for pedestrians and bicyclists crossing the roadway.
  – Islands can provide for the placement of traffic control devices and luminaires.
  – Islands can provide areas within the roadway for landscaping.
- Location of traffic islands can help reduce wrong-way movements at intersections with freeway ramp terminals, especially side-by-side on/off ramps. Intersection islands also have pedestrian accessibility requirements that must be met in design.

I-3. When you use a channelized right-turn lane without an acceleration lane, which of the designs in [Figure 2-7] *is more common?*

Responding state agencies were almost evenly split on this answer, with six agencies stating that the design on the left of Figure 2-7 was their preference, and four indicating that the design on the right was more common. One agency did not answer this question.

a. What are typical site characteristics that led you to select the design you chose?

Characteristics that respondents identified in response to this question varied, with vehicle and pedestrian volumes, intersection size, and available sight distance being the most common. All of the provided characteristics, and the frequency with which they were stated, are listed in Table 3-18. Respondents could choose as many characteristics as they wished, so the sum of the frequencies is greater than the 10 agencies that actually answered Question I-3.

b. What are the site characteristics that would lead you to select the other design?

Responses to this question were similar to those of the previous question. Responses, and the frequency with which they were provided, are listed in Table 3-19.

Table 3-18. Summary of responses to Question I-3a.

Site Characteristic	Frequency (n=10)
Traffic volume	4
Pedestrian accommodation	4
Sight distance for turning drivers	3
Large intersection	3
Design speed	2
Turning radius	2
Intersection angle	2
Available right-of-way	2
Types of vehicles using the intersection	1
Receiving lanes	1
Downstream acceleration lane	1
Signal control	1
Location of traffic control devices	1
No answer	1

I-4. Is there guidance on designing channelizing islands at right-turn lanes not currently included in your state policy or in the AASHTO Green Book *that would be useful? If so, what?*

One agency representative suggested that information on the interaction of right-turn channelization with regard to the impact on pedestrians would be useful. Another respondent said that his/her agency does not have any suggestions for additional guidance, but the bureau responsible for snow removal is opposed to channelization for reasons that have nothing to with the need for channelization. The remaining agencies either had no suggestions (seven respondents) or did not answer the question (two respondents).

I-5. Please list up to three locations in your state with installations that you would consider "best practice" sites.

Respondents from three states provided a total of 10 locations for consideration. The sites are listed in Table 3-20.

Table 3-19. Summary of responses to Question I-3b.

Site Characteristic	Frequency (n=10)
Traffic volume	3
Sight distance for turning drivers	3
Pedestrian accommodation	2
Design speed	2
Intersection angle	2
Receiving lanes	2
Insufficient right-of-way	1
Types of vehicles using the intersection	1
Large intersection	1
Turning radius	1
Downstream acceleration lane	1
Upstream deceleration lane	1
Signal control	1
None/No answer	2

Table 3-20. Summary of responses to Question I-5.

State	City/County	Site
Colorado	(not specified)	SH 391 (MP 0.97)/SH 8
Colorado	(not specified)	SH 391 (MP 4.23)/W. Alameda Ave
Colorado	(not specified)	SH 119 (MP 59.04)/E. 3rd Ave
Florida	Tallahassee	Appalachee Parkway @ Capital Circle NE
Florida	Tampa	State Road 60 @ US 92
Florida	West Palm Beach	State Road 80 @ Military Trail
Maine	Winthrop	Route 202 and Main Street
Maine	Winthrop	Main St. at Routes 41 & 133
Maine	Freeport	Route 136 at Route 125
Maine	Freeport	Route 1 and Desert Road

I-6. Do you have sites that are less than optimal? If so, what improvements do you consider needed and why?

Five respondents offered ideas for improvements based on their experiences. Those responses follow:

- Increase the turning radius to accommodate WB-67.
- Pedestrian treatments where a raised island may not be practical. Currently reviewing shape of islands for Elder Road User program.
- Improve island configuration, sight distance, intersection angle, lane tapers, and acceleration lane.
- Some sites constructed under a previous standard design (using a yield-condition design similar to Figure A) in Question I-3 with a 1:15 closing taper rate on the receiving roadway) have some issues with rear-end crashes. They are typically being refitted with newer standard designs upon reconstruction, but the crash frequency has typically not been high enough to warrant stand-alone projects.
- Some sites with concerns about vehicle/pedestrian interaction. These are reviewed to determine what is causing the concern and if improvements can be made, such as improving the sight distance or other features.

CHAPTER 4

Typical Designs

Introduction

As part of the questionnaire sent to key state transportation agencies, respondents were asked to identify locations with installations that would be considered best-practice sites. These best-practice sites were to demonstrate preferred design treatments for five design categories: island design, deceleration lane design, double left-turn lane design, triple left-turn lane design, and double right-turn lane design. Colorado, Florida, Maine, Minnesota, North Carolina, and Washington all identified locations for consideration.

An underlying question associated with the identification of these locations centered on whether the Green Book provides sufficient guidance for implementing the treatment. Each of the identified sites was examined using aerial imagery to gain a better understanding of the key features of the design, as implemented. A single site considered representative of the design treatment was identified for each of the five design categories. This representative site was examined in detail through a case study approach.

The case studies focused on the key design features associated with the treatment under consideration. To the extent possible, these design features were quantified and referenced to the appropriate standards. A review of each of the completed case studies was then done to determine if the guidance provided by the Green Book would have been sufficient or if supplemental guidance would have been necessary to complete the design. In those cases where supplemental guidance was necessary, this supplementary guidance has been summarized.

Case Studies

Case studies were undertaken to evaluate a typical, best-practice design for five design categories: island design, deceleration lane design, double left-turn lane design, triple left-turn lane design, and double right-turn lane design.

Typical Design: Island

Context

The intersection reviewed for island design is in a suburban area of Lakewood, Colorado, a western suburb of Denver (see Figure 4-1). The primary land use in the area of the intersection is light-density residential, with commercial uses fronting the primary arterials. State Highway 391, also known as S. Kipling Street, is a four-lane divided arterial with a posted speed limit of 45 mph. West Alameda Avenue is a four-lane arterial with a posted speed limit of 45 mph. All right-turning movements at the intersection are provided a deceleration lane, channelization island, and acceleration lane. The channelization islands are raised, curbed islands. The central area of the islands is paved with a colored, textured pavement and contains sidewalks and curb ramps that are depressed and of different texture than the rest of the island.

For this case study, the feature being considered was the island in the northeast quadrant of the intersection (see Figure 4-2).

Design Considerations

According to AASHTO's *A Policy on Geometric Design of Highways and Streets* (*4*), an island is a defined area between traffic lanes used to control vehicle movements and to provide an area for pedestrian refuge and placement of traffic control devices. The *Colorado Roadway Design Guide* (*68*) defines a channelized intersection as an at-grade intersection in which traffic is directed into definite paths by islands. The *Colorado Roadway Design Guide* further identifies the purposes for which islands are generally included in intersection design:

- Separation of conflicts.
- Control of angle of conflict.
- Reduction in excessive pavement areas.

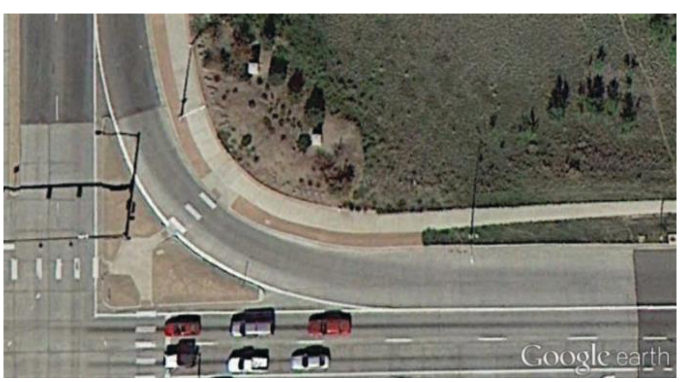
Source: Google Earth™ Mapping Service

Figure 4-1. State Highway 391 (S. Kipling Street) and West Alameda Avenue in Lakewood, Colorado.

- Regulation of traffic and indication of proper use of intersection.
- Arrangements to favor a predominant turning movement.
- Protection of pedestrians (ADA requirements should be considered).
- Protection and storage of turning and crossing vehicles.
- Location of traffic control devices.
- Access control.

The turning roadway is made up of five distinct components: an approach taper, a deceleration lane, the turning roadway curve, an acceleration lane, and a merging taper.

Approach Taper. An approach taper is used to develop the necessary width for the auxiliary lane. This taper allows a driver to recognize that an exclusive lane is being developed and provides a location where some deceleration may occur prior to entering the auxiliary lane. The *Colorado Roadway Design Guide* recommends a taper ratio of 15:1 for a design speed of 50 mph. At this location, an auxiliary lane width of 20 ft was selected to provide for truck movements. Based on the 15:1 taper requirements, 300 ft of approach taper should be provided. At this location, 350 ft of taper was provided.



Source: Google Earth™ Mapping Service

Figure 4-2. Close-up of northeast quadrant.

Deceleration Lane. The *Colorado Roadway Design Guide* points out that it may not be feasible to provide full deceleration lengths on many arterial facilities. As such, it assumes that braking begins "where two-thirds of the lane width is developed" and, thus, the deceleration length begins at that point and extends to the turning roadway curve. Further, a "10 mph differential is commonly considered acceptable on arterial roadways" representing a deceleration of 10 mph within the through lanes and the approach taper. As a continuous-flow turn, it is assumed that the operating speed on the turning roadway curve is 15 mph and that a vehicle entering the deceleration lane will only need to decelerate to 15 mph before the curve, not to a complete stop. When determining deceleration lengths that do not end in a stop condition, the *Colorado Roadway Design Guide* refers the designer to Exhibit 10-73 of the 2004 AASHTO *A Policy on the Geometric Design of Highways and Streets* (*4*) to calculate the required deceleration distance.

At this location, the design speed is 50 mph. Assuming that 10 mph of deceleration occurs on the through lane and within the initial taper area, the design vehicle speed at the beginning of the deceleration lane is 40 mph. Based on Exhibit 10-73, a deceleration distance of 350 ft must be provided to decelerate from 40 mph to 15 mph. This location provides 350 ft of deceleration distance.

Turning Roadway Curve. Based on AASHTO's *A Policy on the Geometric Design of Highways and Streets* (*4*), the "principal controls for the design of turning roadways are the alignment of the traveled way edge and the turning roadway width." The *Colorado Roadway Design Guide* specifies curve design for the inner edge of pavement, turning roadway pavement width, and approximate island size for three design classifications. The design data for an angle of turn of 90 degrees, which matches this location, is shown in Table 4-1. To accommodate a WB-50, Design Classification C is used at this location, resulting in a minimum curve of 65 ft radius, an offset of 6 ft, and terminal curves of 180 ft radius. This design is shown in Figure 4-3.

This design approach permits the right and left wheel tracks of the design vehicle to traverse the traveled way within 2 ft of the edge of the traveled way. While this guidance is generally applicable, designers should always check the turning paths of potential design vehicles to ensure that the final design meets their needs.

Acceleration Lane. The *Colorado Roadway Design Guide* points out that provision for acceleration clear of the through-traffic lanes is a desirable objective on arterial roads and streets.

Table 4-1. Colorado tabular values for treatment of turn lanes (*68*).

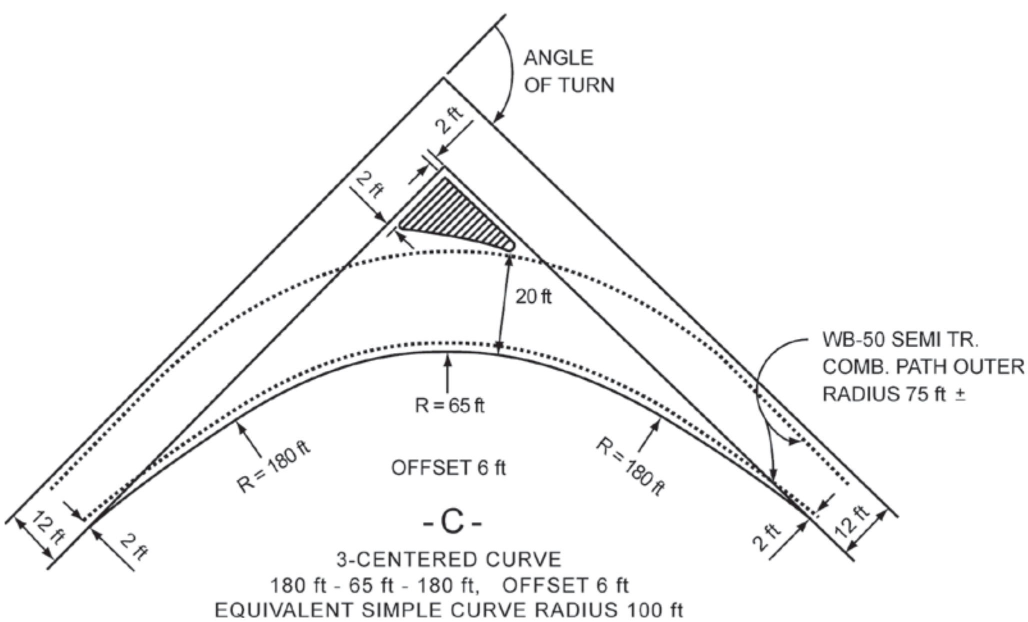
Angle of Turn (degrees)	Design Classification	Three-Centered Curve: Radii (ft)	Three-Centered Curve: Offset (ft)	Width of Lane (ft)	Approximate Island Size (sq ft)
90	A	150-50-150	3.0	14	50
	B	150-50-150	5.0	18	80
	C	180-65-180	6.0	20	125

Notes: A—Primarily passenger vehicles: permits occasional design single-unit truck to turn with restricted clearances.
B—Provides adequately for SU: permits occasional WB-50 to turn with slight encroachment on adjacent traffic lanes.
C—Provides fully for WB-50. Asymmetric three-centered compound curves and straight tapers with a simple curve can also be used without significantly altering the width of roadway or corner island size.

The distance to achieve a safe and comfortable speed at the point of convergence is based on the difference in speed between the operating speed on the turning roadway curve and the operating speed on the arterial. It is assumed that the operating speed on the turning roadway curve is 15 mph. At this location, a parallel-type acceleration lane is provided, which provides operational and safety benefits through additional time for merging vehicles to find an acceptable opening. When determining acceleration lengths that do not begin in a stop condition, the *Colorado Roadway Design Guide* refers the designer to Exhibit 10-70 of the 2004 Green Book to calculate the required acceleration distance.

At this location, the design speed is 50 mph. Based on Exhibit 10-70, an acceleration distance of 660 ft must be provided to accelerate from 15 mph to 39 mph. However, at this location, there is a cross street to the north of the intersection. This physically limits the acceleration distance that can be provided. To accommodate this, the acceleration lane begins as full width and is then tapered to a width of 12 ft, which is then present to the cross-street intersection as an auxiliary lane. The configuration used is 130 ft of full 20-ft width, followed by 170 ft of taper from 20 ft to 12 ft, followed by 200 ft of 12 ft width auxiliary lane. This provides 500 ft of acceleration distance.

Merging Taper. As with the approach taper design, the *Colorado Roadway Design Guide* recommends a taper ratio of 15:1 for a design speed of 50 mph. Due to the use of the drop lane described above, the acceleration lane only needs to taper from 20 ft to 12 ft. The configuration used is 130 ft of



Source: *A Policy on Geometric Design of Highways and Streets* (2004) by the American Association of State Highway and Transportation Officials, Washington, D.C. Used by permission.

Figure 4-3. Minimum turning roadway designs with corner islands at urban locations (WB-50 design vehicle) (4).

full 20 ft width, followed by 170 ft of taper from 20 ft to 12 ft, followed by 200 ft of 12 ft width auxiliary lane.

Corner Island Delineation. The Green Book states that “the island in all instances should be about 2 ft outside the traveled way edges.” It further provides guidance for the delineation of islands to ensure that the approach nose of a curbed island is conspicuous to approaching drivers and clear of vehicle paths so that drivers will not shy away from the island. Details on this guidance are shown in Figure 4-4.

The guidance calls for the offset of the approach nose (4 to 6 ft) to be greater than the offset to the face of the curbed island (2 to 3 ft). At this location, the offset of the approach nose is 4 ft, while the offset to the face of the curbed island is 2 ft. This allows the island to be gradually widened to its full width past the approach nose.

In addition, the approach nose should be provided with devices to give advance warning to drivers of the island, especially for nighttime operation. In a northern climate, it is also important to clearly delineate the island location because snowfall may obscure the curbed edge. To account for both of these concerns at this location, post-mounted reflectors are positioned on the island near the approach nose, merging nose, and approach corner to help delineate the island location.

It is further recommended that all curbed island corners should be rounded with appropriate curve radii for visibility and construction simplicity. It recommends that the approach nose and merging nose use a 2- to 3-ft radius and the approach corner use a radius of 2 to 5 ft. At this location, the approach nose uses a 2.5-ft radius, the merging nose a 2-ft radius, and the approach corner a 4-ft radius.

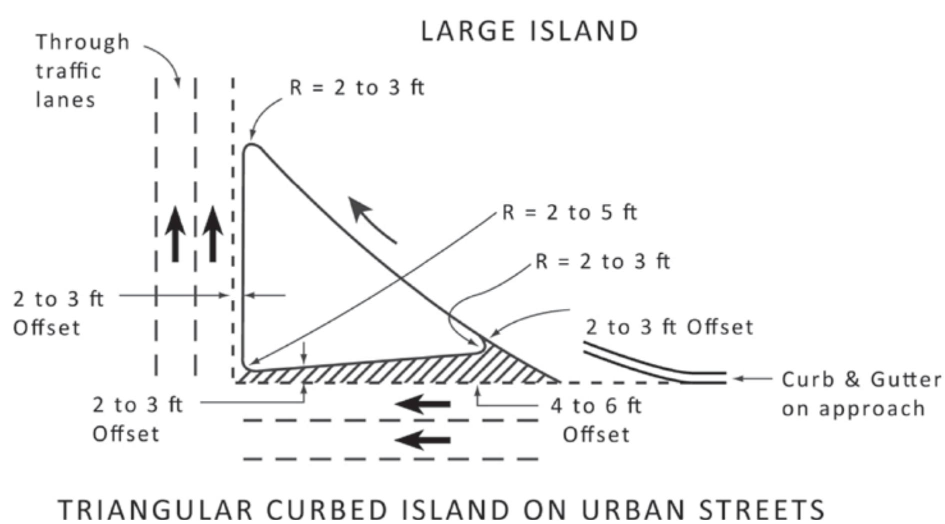
Additional Considerations

Pedestrians. It is preferable that passages for pedestrians and wheelchairs on raised islands be cut-through or at-grade installations. This better facilitates guidance for visually impaired pedestrians and eliminates the need to traverse additional grades for wheelchairs. At this location, the pedestrian crossings within the channelized islands, while not at grade, are depressed as compared to the surrounding island. In addition, they are of different texture then other parts of the island.

Reduced Visibility. The *Colorado Roadway Design Guide* points out that curbed islands can be difficult to see at night. As such, intersections using curbed islands should provide fixed-source lighting or appropriate delineation. At this location, both are provided.

Snow. An additional consideration for islands in northern climates is their potential impact on snow removal activities. *Colorado Roadway Design Guide* recommends that designers consider the implementation of plowable end treatments. While this was not done at this location, post-mounted reflectors are installed at each of the three island corners (approach nose, merging nose, and approach corner) to provide clear delineation of the island dimensions even during inclement weather.

Functional Intersection Area. The *Colorado Roadway Design Guide* lists functional intersection area as one of the five basic elements that enter into design considerations of intersections. The functional intersection area includes areas both upstream and downstream of the intersection.



Source: *A Policy on Geometric Design of Highways and Streets* (2004) by the American Association of State Highway and Transportation Officials, Washington, D.C. Used by permission.

Figure 4-4. Details of corner island design for turning roadways (urban location, large island) (4).

Source: Google Earth™ Mapping Service

Figure 4-5. State Route 42 (W. Academy Street) and Coley Farm Road in Fuquay-Varina, North Carolina.

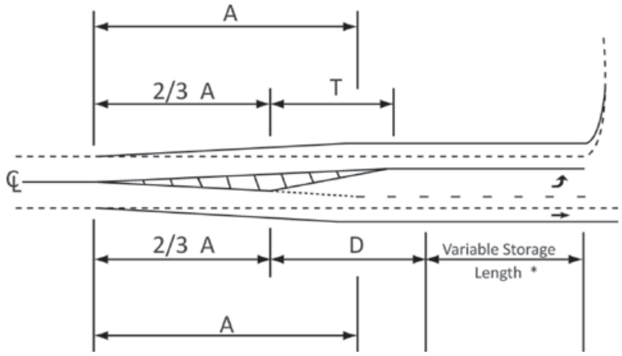


Figure 4-6. Recommended treatment for turn lanes (69).

While this area is variable, based on local conditions, it is accepted that vehicles leaving a major roadway disrupt traffic flow, which affects overall operation and safety. Thus, good access management is critical in planning and designing a roadway so that it performs according to its functional classification. At this location, there is no access within the intersection functional area in three of the four quadrants.

Typical Design: Deceleration Lane

Context

The intersection reviewed for deceleration is in a rural area to the west of Fuquay-Varina, North Carolina (see Figure 4-5). The primary land use in the area of the intersection is low-density residential. State Route 42, also known as W. Academy Street, is a two-lane undivided highway. This intersection along State Route 42 is where the cross-section transitions from a curb-and-gutter section east of the intersection to unpaved shoulders west of the intersection. West of the intersection, State Route 42 consists of two 11-ft lanes with a posted speed of 45 mph. East of the intersection, it consists of two 15-ft lanes, also with a posted speed of 45 mph.

Coley Farm Road is a two-lane arterial with a posted speed of 35 mph north of the intersection and an assumed speed of 35 mph (based on North Carolina's general statutes) south of the intersection as no posted speed was found there. This case study examined the east and west leg of the intersection and focused on the design of the deceleration lanes provided for turning traffic. The design speed for State Route 42 is 50 mph.

Design Considerations

According to the 2004 AASHTO *A Policy on Geometric Design of Highways and Streets* (4), "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 *North Carolina Roadway Design Manual* (69) considers some deceleration within the taper area of the developing auxiliary lane and, thus, reduces the required deceleration distances appropriately. The design of a deceleration lane based on the *North Carolina Roadway Design Manual* is presented in Table 4-2 and Figure 4-6.

Table 4-2. North Carolina tabular values for treatment of turn lanes (*69*).

Design Speed (mph)	Posted Speed (mph)	Minimum Deceleration Length (D)	Desirable Deceleration Length (D)	Bay Taper Length (T)	Approach/Departure Taper (A)
30	≤ 25	100'	150'	75'	$A = WS^2/60$ (If S ≤ 40 mph)
35	30	100'	150'	75'	A = WS (If S > 40 mph)
40	35	150'	200'	100'	S = Design Speed
45	40	150'	250'	100'	W = Width of Lateral Shift
50	45	150'	300'	100'	Storage length for waiting vehicles should be calculated based on the latest version of the Highway Capacity Manual or Policy on Street and Driveway Access to North Carolina Highways.
55	50	200'	500'	150'	
60	55	200'	575'	200'	

Intersection East Leg

The development of the left-turn deceleration lane for traffic traveling westbound on State Route 42 was done symmetrically about the centerline at this location. The roadway cross-section approaching the intersection area consists of two 15-ft lanes (excluding curb and gutter). This is then developed into three 11-ft lanes within the intersection (excluding curb and gutter)—a through lane in each direction and a westbound left-turn lane. To accommodate the intersection lanes, the pavement must be widened by 1.5 ft on each side of the centerline (W=1.5 ft).

Table 4-2 provides formulas for calculating the approach taper lengths based on the design speed (S) and width of lateral shift (W). Because the design speed is greater than 40 mph, the formula A=WS with values of S=50 and W=1.5 was used to calculate a value for the approach taper (A) of 75 ft. Further, based on Figure 4-6, the dimensions for the channelization of the deceleration lane can be identified using a value of ⅔ A (50 ft) and T (100 ft). However, at this location, a value of T=75 ft was used.

Intersection West Leg

The development of both the left-turn and right-turn deceleration lanes for traffic traveling eastbound on State Route 42 was done symmetrically about the centerline at this location. The roadway cross-section approaching the intersection area consists of two 11-ft lanes. This is then developed into four 10-ft lanes within the intersection—a through lane in each direction, an eastbound right-turn lane, and an eastbound left-turn lane. To accommodate the intersection lanes, the pavement must be widened by 9 ft on each side of the center line (W=9 ft).

Table 4-2 provides formulas for calculating the approach taper lengths based on the design speed (S) and width of lateral shift (W). Because the design speed is greater than 40 mph, the formula A=WS with values of S=50 and W=9 was used to calculate a value for the approach taper (A) of 450 ft. Further, based on Figure 4-6, the dimensions for the channelization of the deceleration lane can be identified using a value of ⅔ A (300 ft) and T (100 ft). However, at this location, a value of T=75 ft was used.

Additional Considerations

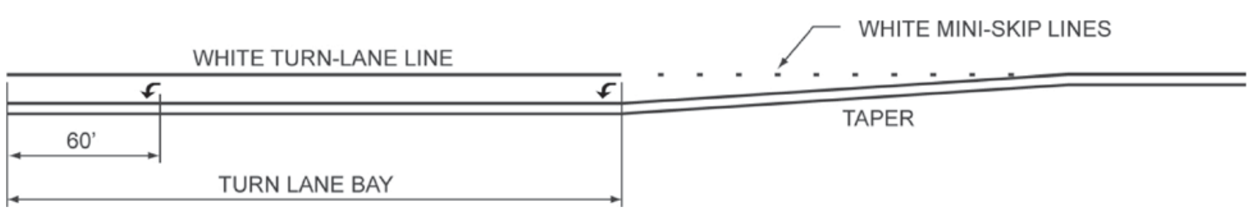
The North Carolina DOT also maintains a series of Roadway Standard Drawings. When providing for deceleration lanes, the standard drawing for Pavement Markings: Turn Lanes specifies their treatment within the intersection area (see Figure 4-7). Under this guidance, skip striping is used to outline the entry taper area. Past this point, solid striping is used to delineate the auxiliary lane. At this location, this results in the skip striping being used on both the east and west leg of the intersection for the left-turn auxiliary lanes from the beginning of the entering taper to 75 ft past the start of the taper. At this point, all striping reverts to solid. In addition, on the west leg of the intersection, skip striping is used to delineate the entry taper for the right-turn lane from the beginning of the entering taper to 300 ft past the entering taper. At this point, all striping reverts to solid.

Typical Design: Double Right-Turn Lanes

Context

The intersection reviewed for double right-turn lanes is in an urbanized area of Wellington, Florida (see Figure 4-8). The primary land use in the area of the intersection is residential. The southwest quadrant of this intersection, however, is a large regional mall—the Mall at Worthington Green—and there is commercial development facing the major arterials. State Road 7, also known as Range Line Road, runs approximately north/south at this location. The cross-section of State Road 7 is an eight-lane divided arterial with a bike lane in each direction. It has a posted speed limit of 50 mph. State Road 882, also known as Forest Hill Boulevard, runs approximately east/west at this location. It is a six-lane divided arterial with a bike lane in each direction. It has a posted speed limit of 50 mph to the east of the intersection and 45 mph to the west of the intersection.

This case study examined the west leg of the intersection and focused on the design of the double right-turn lanes provided for traffic turning from eastbound State Road 882 to southbound State Road 7 (see Figure 4-9). The intersection was recently upgraded with a design speed of 60 mph.



Source: *North Carolina Roadway Standard Drawings,* North Carolina Department of Transportation, 2006.

Figure 4-7. Pavement markings for turn bays 125 to 200 ft in length.

Source: Google Earth™ Mapping Service

Figure 4-8. State Road 7 (Range Line Road) and State Road 882 (Forest Hill Boulevard) in Wellington, Florida.

Design Considerations

According to the Florida *Manual of Uniform Minimum Standards for Design, Construction and Maintenance for Streets and Highways* (referred to as the Florida Greenbook) (*70*), "Storage (or deceleration lanes) to protect turning vehicles should be provided, particularly where turning volumes are significant" to accommodate speed change and maneuvering of turning traffic. Based on the projected demand for turns on this leg of the intersection, double right-turn lanes were provided.

Auxiliary Lane Design

The *Florida Intersection Design Guide* (*71*) details the requirements for the design of the auxiliary lanes to accommodate turn lanes. Based on this guidance, the auxiliary lane is defined by three key components: entering taper, deceleration length,

Source: Google Earth™ Mapping Service

Figure 4-9. Close-up of double right-turn lanes.

and storage length. The minimum lengths associated with these key components are outlined in Table 4-3 and Figure 4-10. The guidance focuses on left-turn examples, but the approach is the same for right-turn auxiliary lanes.

The entering taper requirement is 100 ft for double left-turn lanes (see Figure 4-10). The short taper length is meant to "provide approaching road users with positive identification" of the upcoming auxiliary lane and to provide for the greatest length of full-width auxiliary lanes to the extent possible. At this location, a taper of 100 ft is provided.

To account for the actual path that road users will use to access the auxiliary lanes based on speed conditions, two clearance distances (L_1 and L_3) are identified (see Figure 4-10). These distances also help identify the locations at which to begin the striping of the lane lines for the auxiliary lanes. Based on a design speed of 60 mph, L_1 has a value of 145 ft and L_3 has a value of 230 ft. At this location, the striping for both turn lanes begins at 230 ft from the beginning of the entering taper. The outside turn lane striping did not begin at 145 ft due to the requirements associated with providing for bike lanes discussed below.

The total deceleration length (shown as L in Figure 4-10) is "that needed for a safe and comfortable stop from the design speed of the highway." This distance is made up of the clearance distance, L_1, and the brake-to-stop distance, L_2. The clearance distance identifies the point at which vehicles have entered the auxiliary lane. In urbanized areas, it is assumed a turning vehicle decelerates by 10 mph while traversing the through lane and this is clearance distance. However, for design

Table 4-3. Minimum deceleration lengths based on design speed (*71*).

Turn Lanes—Curbed and Uncurbed Medians								
			Urban Conditions			Rural Conditions		
Design Speed (mph)	Entry Speed (mph)	Clearance Distance L1 (ft)	Brake-To-Stop Distance L2 (ft)	Total Decel. Distance L (ft)	Clearance Distance L3 (ft)	Brake-To-Stop Distance L2 (ft)	Total Decel. Distance L (ft)	Clearance Distance L3 (ft)
35	25	70	75	145	110	----	----	----
40	30	80	75	155	120	----	----	----
45	35	85	100	185	135	----	----	----
50	40/44	105	135	240	160	185	290	160
55	48	125	----	----	----	225	350	195
60	52	145	----	----	----	260	405	230
65	55	170	----	----	----	290	460	270

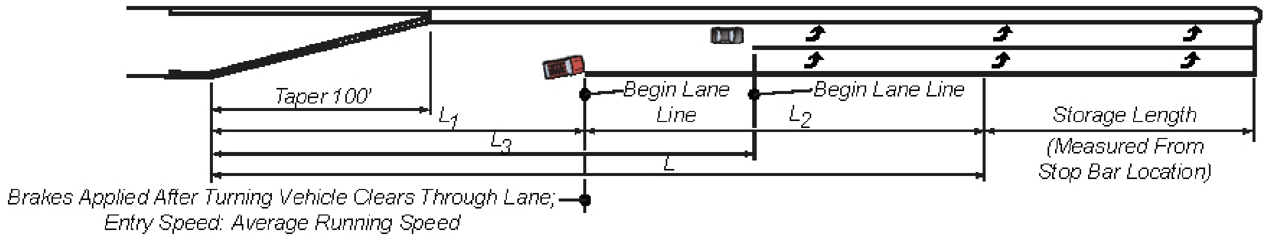


Figure 4-10. Double left-turn with raised separation (71).

speeds over 50 mph, the average running speed is used. Thus, for a design speed of 60 mph, the assumed entry speed is 52 mph. The brake-to-stop distance then represents the distance to bring a vehicle to a stop at the assumed entry speed. At this location, L_1 has a value of 145 ft and L_2 has a value of 260 ft, resulting in a total deceleration length, L, of 405 ft.

Finally, the storage length accounts for the distance necessary to accommodate the number of vehicles likely to accumulate during a critical period. It is assumed that when two turning lanes are used, the storage length required is approximately half of what would be necessary for a single lane (note: not all states assume an even distribution of queue between the two provided turn lanes). At this location, 220 ft of queue storage is provided in each turn lane, which represents a total storage length of 440 ft.

Bike Lane Design

When providing for the through movement of a bicycle lane at a major intersection in an urbanized area with curb and gutter, the Florida Greenbook specifies the location and treatment of that bicycle lane within the intersection area (see Figure 4-11). The bicycle lane is maintained between the through and turning vehicle lanes. Under this guidance, skip striping is used to outline the area where the bicycle and right-turning traffic weaving area is located. When this is

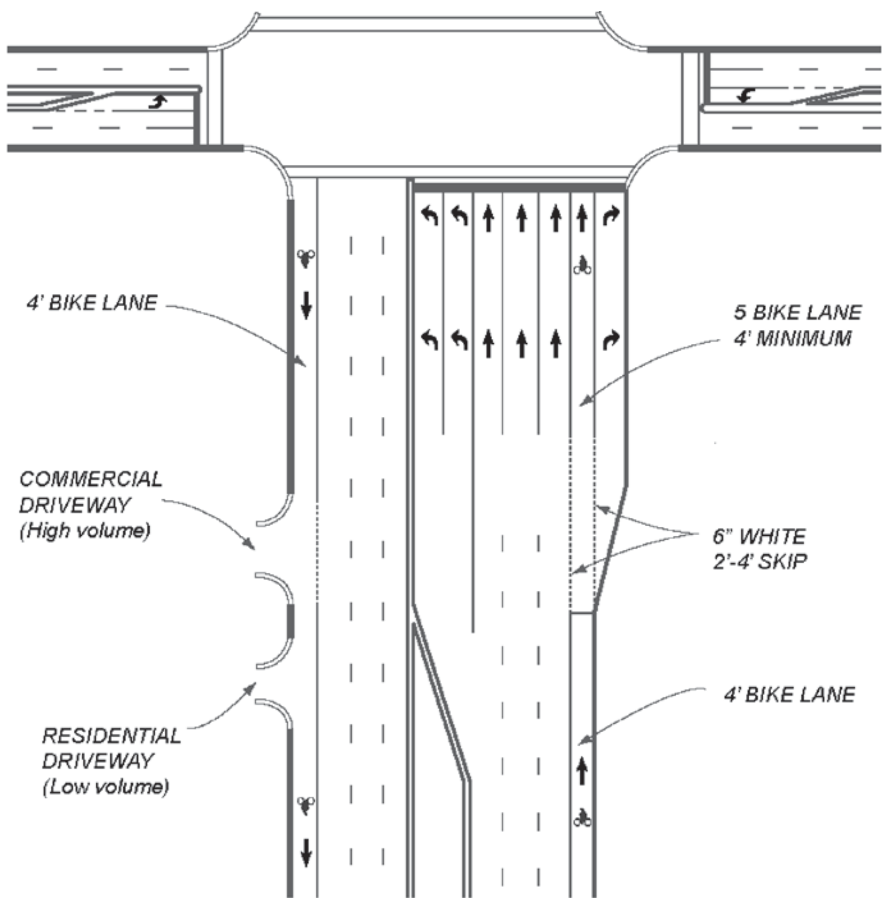


Figure 4-11. Major intersection with separate right-turn lane urban typical section (curb and gutter) (70).

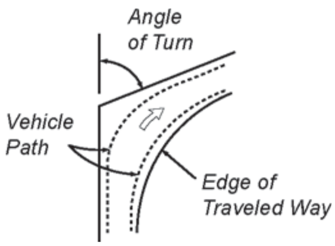


Figure 4-12. Reference turn angle for turning roadway designs (71).

Source: Google Earth™ Mapping Service

Figure 4-13. State Road 7 (Range Line Road) and State Road 704 (Okeechobee Boulevard) in West Palm Beach, Florida.

applied to double turn lanes, this skip striping is extended to the beginning of the outermost turn lane. At this location, this results in the skip striping being used from the beginning of the entering taper to 230 ft past the entering taper. At this point, all striping reverts to solid.

Turning Radii Design

The Florida Greenbook specifies that the design of corner radii should be based on selected design vehicles. According to the guidance, in an urban area the designer must balance several key issues: the needs of the road users using them, the amount of right-of-way available, the angle of turn between intersection legs, the number of pedestrians using the crosswalk, the width and number of lanes on the intersecting street, and the speeds on each street. The minimum-edge-of-traveled-way design is based on the angle of turn and design vehicle (see Figure 4-12). At this location, the angle of turn is 90 degrees and the design vehicle is a WB-62. Based on Table 4-4, the symmetric curve radii for a three-centered compound curve are 400-70-400 ft with an offset of 10 ft.

Additional Considerations

When double right-turn lanes are provided, the design must provide intersection turning radii to accommodate vehicles turning two abreast. In Florida, most intersections on the state highway system must be able to accommodate, at a minimum, an SU truck and passenger car turning simultaneously. Depending on the design vehicles selected, the width of the receiving throat may need to be expanded to accommodate the necessary turning radii. At this location, no additional widening was necessary.

Based on the turning template analysis performed to determine if additional lane widening was necessary for the receiving throat, the delineation between the two swept paths can be established. To provide positive guidance to road users, this delineation is often physically marked out through the intersection using skip striping. These turn guide lines in Florida are specified as a 2-ft skip with a 4-ft gap dotted line. At this location, skip striping was provided.

Typical Design: Triple Left-Turn Lanes

Context

The triple left-turn lane intersection is in an urbanized area to the west of West Palm Beach, Florida (see Figure 4-13). The surrounding area is dominated by residential uses, with retail spaces along major arterial corridors. State Road 7, also known as Range Line Road, runs approximately north/south

Table 4-4. Edge-of-traveled-way design for turns at intersections (*71*).

Angle of Turn (degrees)	Design Vehicle	3-Centered Compound			
		Curve Radii (ft)	Symmetric Offset (ft)	Curve Radii (ft)	Asymmetric (ft)
90	P	100-20-100	2.5	----	----
	SU	120-40-120	2.0	----	----
	WB-40	120-40-120	5.0	120-40-200	2.0-6.5
	WB-50	180-60-180	6.5	120-40-200	2.0-10.0
	WB-62	400-70-400	10.0	160-70-360	6.0-10.0
	WB-67	440-65-440	10.0	200-70-600	1.0-11.0
	WB-100T	250-70-250	4.5	200-70-300	1.0-5.0
	WB-109D	700-110-700	6.5	100-95-550	2.0-11.5

at this location. To the north of this intersection, State Road 7 is a two-lane undivided arterial. However, as it approaches the intersection it widens to a four-lane divided arterial and, south of the intersection, widens again to a six-lane divided arterial. It has a posted speed limit of 45 mph and a bike lane in each direction. State Road 704, also known as Okeechobee Boulevard, runs approximately east/west at this location. It is an eight-lane divided arterial with a posted speed limit of 50 mph and a bike lane in each direction (see Figure 4-14).

This intersection provides for triple left turns on two of the approaches—northbound State Road 7 and westbound State Road 704. Because the cross-section of State Road 7 changes from a six-lane to four-lane section north of the intersection, one of the through lanes is dropped at the intersection (sometimes referred to as a trap lane) as a left-turn lane for the south leg of the intersection forming one of the lanes of the triple left-turn provided on that approach (a Type B Triple Left Turn). On the east leg of the intersection (see Figure 4-15) for the westbound State Road 704 approach, the triple left turn is fully developed as auxiliary lanes (a Type A Triple Left Turn). As such, this case study focused on the east leg of the intersection and the triple left turn provided for traffic traveling from westbound State Road 704 to southbound State Road 7. The design speed for this approach is 60 mph (10 mph over the posted speed).

Design Considerations

The *Florida Intersection Design Guide* (*71*) details the requirements for the design of the auxiliary lanes to accommodate both single and double left-turn lanes. While there

Source: Google Earth™ Mapping Service

Figure 4-14. Close-up of triple left-turn lanes from northbound approach.

Source: Google Earth™ Mapping Service

Figure 4-15. Close-up of westbound approach.

is no specific design guidance on development of triple left-turn lanes, the concepts associated with both the single and double left-turn lanes can be adapted. Under this guidance, the auxiliary lane is defined by three key components: entering taper, deceleration length, and storage length. The minimum lengths associated with these key components are outlined in Table 4-3 and Figure 4-10.

The entering taper requirement for a single left-turn lane is 50 ft, while it is 100 ft for double left-turn lanes. The short taper length is meant to "provide approaching road users with positive identification" of the upcoming auxiliary lane and to provide for the greatest length of full-width auxiliary lanes to the extent possible. This concept has been expanded for use with a triple left-turn condition by expanding the taper length to 150 ft at this location.

To account for the actual path that road users will use to access the auxiliary lanes based on speed conditions, two clearance distances (L_1 and L_3) are identified (see Figure 4-10). These distances also help identify where to begin the striping of the lane lines for the auxiliary lanes. Based on a design speed of 60 mph, L_1 has a value of 145 ft and L_3 has a value of 230 ft. At this location, the striping for the outside turn lane begins at 175 ft and for both the middle and the inside turn lane begins at 230 ft from the beginning of the entering taper.

The total deceleration length (shown as L in Figure 4-10) is "that needed for a safe and comfortable stop from the design speed of the highway." This distance is made up of the clearance distance, L_1, and the brake-to-stop distance, L_2. The clearance distance identifies the point at which vehicles have entered the auxiliary lane. In urbanized areas, it is assumed a turning vehicle decelerates by 10 mph while traversing the through lane and this clearance distance. However, for design speeds over 50 mph, the average running speed is used. Thus, for a design speed of 60 mph, the assumed entry speed is 52 mph. The brake-to-stop distance then represents the distance to bring a vehicle to a stop at the assumed entry speed. At this location, L_1 has a value of 145 ft and L_2 has a value of 260 ft, resulting in a total deceleration length, L, of 405 ft.

Finally, the storage length accounts for the distance necessary to accommodate the number of vehicles likely to accumulate during a critical period. It is assumed that when two turning lanes are used, the storage length required is approximately half of what would be necessary for a single lane (note: not all states assume an even distribution of queue between the two provided turn lanes). Expanding this approach to represent triple left-turns at this location, 285 ft of queue storage is provided in each turn lane, which represents a total storage length of 855 ft.

Additional Considerations

Because of the added length of the entry taper used to develop the triple left-turn auxiliary lanes, the entry area into these auxiliary lanes is larger than typical. To ensure that drivers receive positive guidance in this area, additional skip striping was used from the beginning of the entry taper to the beginning of the outside left-turn lane line. This skip striping provides a clear delineation of the inside through lane within this transition area.

When triple left-turn lanes are provided, the design must provide intersection turning radii to accommodate vehicles turning simultaneously. For double left-turn lanes in Florida, most intersections on the state highway system must be able to accommodate at a minimum an SU truck and passenger car turning simultaneously. Depending on the design vehicles selected, the width of the receiving throat may need to be expanded to accommodate the necessary turning radii. At this location, no additional widening was necessary.

Based on the turning template analysis performed to determine if additional lane widening was necessary for the receiving throat, the delineation between the swept paths can be established. To provide positive guidance to road users, this delineation is often physically marked out through the intersection using skip striping. These turn guidelines in Florida are specified as a 2-ft skip with a 4-ft gap dotted line. At this location, skip striping was provided.

The *Florida Intersection Design Guide* (*71*) identifies that intersections are defined by both their physical and functional areas. The functional area of an intersection extends both upstream and downstream from the physical area and includes auxiliary lanes and associated channelization. Driveways should not be within the functional area to improve both operational and safety characteristics of the intersection. At this location, access is limited to roadway segments outside the functional area.

Typical Design: Double Left-Turn Lanes

Context

This intersection is in a rapidly urbanizing corridor southeast of Tallahassee, Florida (see Figure 4-16). Capital Circle is a non-limited access beltway that encircles approximately three-quarters of Tallahassee on the west, south, and east. This section of the arterial, Capital Circle SE, is also the alignment of US Route 319 and Florida State Road 261 and runs approximately north/south at this location. Capital Circle SE cross-section is three through lanes and a bike lane in each direction with a posted speed of 45 mph. Blair Stone Road has a cross-section of two through lanes and a bike lane in each direction with a posted speed limit of 35 mph.

Source: Google Earth™ Mapping Service

Figure 4-16. Capital Circle SE and Blair Stone Road, in Tallahassee, Florida.

This case study examined the east leg of the intersection and focused on the design of the double left-turn lanes provided for traffic turning from westbound Blair Stone Road to southbound Capital Circle SE. The design speed for this approach is 40 mph (5 mph over the posted speed). However, all approaches at the intersection provide double left-turn lanes.

Design Considerations

According to the Florida Greenbook, "Storage (or deceleration lanes) to protect turning vehicles should be provided, particularly where turning volumes are significant." Based on the projected demand for turns at this intersection, double left-turn lanes were provided on each of the approaches.

The *Florida Intersection Design Guide* (*71*) details the requirements for the design of the auxiliary lanes to accommodate the double left-turn lanes. Based on this guidance, the auxiliary lane is defined by three key components: entering taper, deceleration length, and storage length. The minimum lengths associated with these key components are outlined in Figure 4-10 and Table 4-3.

The entering taper requirement is 100 ft for double left-turn lanes (see Figure 4-10). The short taper length is meant to "provide approaching road users with positive identification" of the upcoming auxiliary lane and to provide for the greatest length of full-width auxiliary lanes to the extent possible. At this location, a taper of 100 ft is provided.

To account for the actual path that road users will use to access the auxiliary lanes based on speed conditions, two clearance distances (L_1 and L_3) are identified (see Figure 4-10). These distances also help identify the locations at which to begin the striping of the lane lines for the auxiliary lanes. Based on a design speed of 40 mph, L_1 has a value of 80 ft and L_3 has a value of 120 ft. At this location, the striping for the outside turn lane begins at 80 ft and for the inside turn lane begins at 120 ft from the beginning of the entering taper.

The total deceleration length (shown as L in Figure 4-10) is "that needed for a safe and comfortable stop from the design speed of the highway." This distance is made up of the clearance distance, L_1, and the brake-to-stop distance, L_2. The clearance distance identifies the point at which vehicles have entered the auxiliary lane. In urbanized areas, it is assumed a turning vehicle decelerates by 10 mph while traversing the through lane and this clearance distance. The brake-to-stop distance then represents the distance to bring a vehicle to a stop at the assumed entry speed. At this location, L_1 has a value of 80 ft and L_2 has a value of 75 ft, resulting in a total deceleration length, L, of 155 ft.

Finally, the storage length accounts for the distance necessary to accommodate the number of vehicles likely to accumulate during a critical period. It is assumed that when two turning lanes are used, the storage length required is approximately half of what would be necessary for a single lane (note: not all states assume an even distribution of queue between the two provided turn lanes). At this location, 65 ft of queue storage is provided in each turn lane, which represents a total storage length of 130 ft.

Additional Considerations

When double left-turn lanes are provided, the design must provide intersection turning radii to accommodate vehicles turning two abreast. In Florida, most intersections on the state highway system must be able to accommodate at a minimum an SU truck and passenger car turning simultaneously (see Figure 4-17). Depending on the design vehicles selected, the width of the receiving throat may need to be expanded to accommodate the necessary turning radii. At this location, no additional widening was necessary.

Based on the turning template analysis performed to determine if additional lane widening was necessary for the receiving throat, the delineation between the two swept paths can be established. To provide positive guidance to road users, this delineation is often physically marked out through the intersection using skip striping. These turn guidelines in Florida

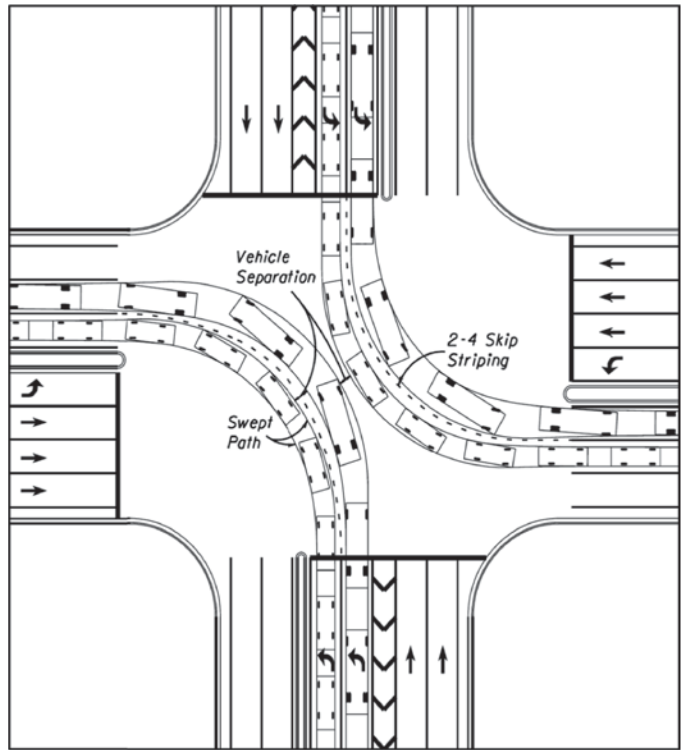


Figure 4-17. SU truck and passenger car turning simultaneously (71).

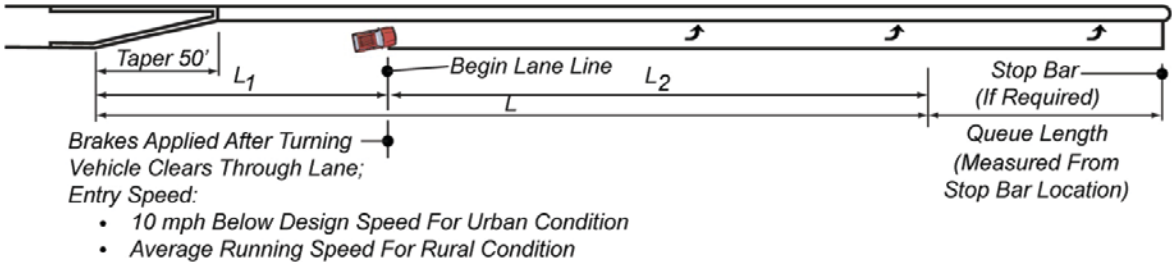
are specified as a 2-ft skip with a 4-ft gap dotted line. At this location, skip striping was provided.

At this location, additional emphasis was undertaken to identify pedestrian crossing locations with pavement that has been both textured (to appear as brick) and colored.

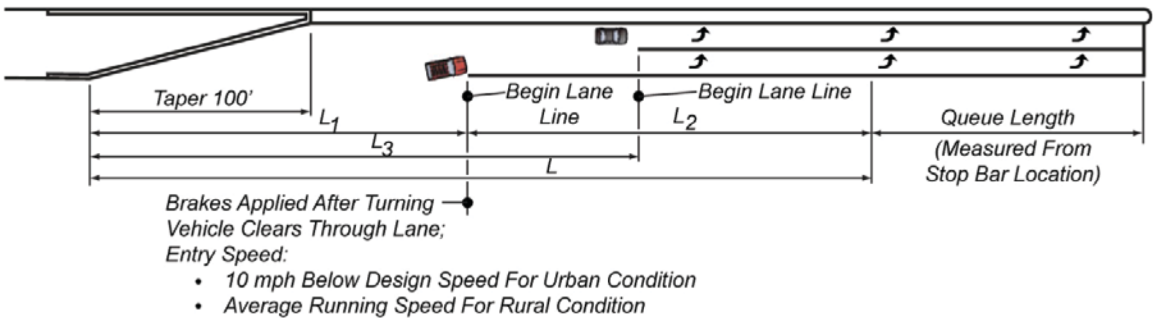
Supplementary Guidance

When the case studies were examined to determine if supplementary guidance above and beyond that in the 2011 version of the Green Book would be beneficial, it was determined that no supplementary guidance was necessary for island design, only minor supplementary guidance was necessary for multiple turn lanes to address the concept of a clearance distance, and supplementary guidance was needed for development of deceleration lanes for undivided roadways. This supplementary guidance is summarized below.

Multiple Turn Lanes. Section 9.7 Auxiliary Lanes of the 2011 Green Book (*2*) provides needed guidance on identifying the functional area of an intersection and determining the deceleration length necessary for developing auxiliary lanes. This section also provides guidance on taper lengths and examples of taper design for development of the auxiliary lanes. Further, this section provides guidance on when double left-turn lanes should be implemented (where left-turn



Single Left-Turn Lane



Double Left-Turn Lane

Figure 4-18. Components of an auxiliary lane (71).

volumes exceed 300 veh/hr) and how to account for offtracking and swept path widths and provides suggestions on the use of skip striping through the intersection for positive guidance. However, this section does not discuss the development of auxiliary lane pavement markings for use in multiple turn lane configurations.

Although this may be viewed as a pavement marking issue, successful implementation through proper location of these pavement markings can affect the operational characteristics of the multiple turning lane configuration. Further, the 2009 *Manual on Uniform Traffic Control Devices* (*72*) does not provide sufficient guidance for determining the beginning of turn lane lines within the transition area of the auxiliary lane. Without positive guidance for drivers within the approach transition to the auxiliary lane, proper vehicle alignment is more difficult, causing more turbulent flow, and, thus, may affect operational characteristics.

The information presented in the 2011 Green Book identifies the full deceleration length as the sum of the taper distance to begin deceleration and complete lateral movement and the distance traveled to complete deceleration to a stop. Florida DOT has identified a clearance distance to account for the actual path that road users will use to access the auxiliary lane based on speed conditions. To account for this, it is suggested that Figure 9-49, Table 9-22, and the text for the subsection Taper Length of the 2011 Green Book be updated to identify and quantify this clearance distance.

Figure 9-49 of the Green Book should be updated to show both single- and double-lane auxiliary lanes, instead of only a single-lane auxiliary lane. The complexity of this figure should be increased to identify the clearance distance. As shown in Figure 4-18, the clearance distances are called out as L_1 and L_3 and represent the location recommended for the beginning of lane line markings for the auxiliary lane.

FDOT also provides guidance on the minimum clearance distances and deceleration distances based on design speed of the roadway (see Table 4-3). However, the values presented are not as conservative as those presented in the Green Book. A direct comparison can be made for design speeds of 40 mph, 50 mph, and 60 mph (see Table 4-5) between AASHTO, North Carolina Department of Transportation (NCDOT), and FDOT minimum deceleration lengths. This shows that FDOT full deceleration distances are 46 to 77% higher than AASHTO values, and NCDOT values are 5 to 40% higher when compared to AASHTO values. As such,

Table 4-5. Comparison of AASHTO, NCDOT, and FDOT full deceleration lengths.

Speed (mph)	Deceleration Distance (ft)		
	Green Book (*2*)	NCDOT (*69*)	FDOT (*70*)
30	160	150	
35		150	145
40	275	200	155
45		250	185
50	425	300	240/290
55		500	350
60	605	575	405
65			460
70	820		

identifying values for use in quantifying the clearance distance are not clear. The text presented in the 2011 Green Book for the subsection, Taper Length, could be updated to identify the concept of a clearance distance, but further investigation will be necessary to identify appropriate clearance distance values for use in updating Table 9-22 of the Green Book.

Deceleration Lane Design. Section 9.7, Auxiliary Lanes, of the 2011 Green Book provides guidance on developing auxiliary lanes. However, the illustrated applications focus on those that occur on roadways that provide a median of sufficient width to accommodate the development of the auxiliary lane. Additional guidance is necessary for illustrating the development of deceleration lanes for undivided roadways, resulting in a flared intersection. Although the current guidance outlines the general characteristics of a flared intersection in Section 9.3.2, Four-Leg Intersection, no specific guidance is given on how to provide the necessary widening. To account for this, a subsection on flared intersections could be added under Section 9.7 Auxiliary Lanes; the material for this subsection could be based on the corresponding information from the *North Carolina Roadway Design Manual* (*69*) described previously in this chapter.

CHAPTER 5

Double Left-Turn Lane Field Study

Background

The objective of NCHRP Project 03-102 was to recommend improvements to the guidance provided in the AASHTO *Policy on Geometric Design of Highways and Streets* (commonly known as the Green Book) (*2*).

Task 1 was to identify the state of design practice for auxiliary lanes at intersections by gathering and synthesizing information on existing practices and research. Task 2 determined the issues that merit further study to validate, enhance, and expand current Green Book guidance. Task 3 produced the interim report and a Phase II work plan, which includes suggested research efforts for the remaining tasks. An area suggested for additional research was on multiple left-turn lanes, such as double left-turn lanes or triple left-turn lanes (TLTL).

Green Book Review on Double or Triple Left-Turn Lanes (Subsection within 9.7.3)

The Green Book section on double or triple left-turn lanes presents information about double and triple left-turn lanes, including advice on offtracking and swept path width. This section is new to the 2011 Green Book. Several general statements are made regarding multiple turn lanes. Table 5-1 lists Green Book statements along with potential research needs.

Item 5 can be addressed with a reference to the *Manual on Uniform Traffic Control Devices* (*72*), Section 3B.08 (Extensions through Intersections or Interchanges). Figure 3B-13 (D) in the MUTCD includes an example of typical dotted line markings to extend lane line markings into the intersection for double left-turn lanes.

A recent TxDOT study (*52*), the FHWA *Signalized Intersection: Informational Guide* (*7*), and a Florida study (*73*) provide guidance for multiple left-turn lanes and/or double right-turn lanes. These references could be used to develop guidance statements for the Green Book. Therefore, the research team suggested that efforts within NCHRP Project 03-102 focus on items 1, 2, and 3 in Table 5-1. The project panel instructed the research team, at the conclusion of Phase I, to focus on the operations of double left-turn lanes.

Study Selection

The goal of this study was to determine the effects of geometric characteristics on operations, as measured using saturation flow rate and lane utilization, for double left-turn lanes. The geometric variables that were the focus of this study were receiving leg width, left-turn lane width, and downstream friction location (type and distance). A friction location is a roadway feature that could affect the behavior of left-turning vehicles within the receiving leg. Examples of friction locations include bus stops, driveways, and exits from channelized right turns.

Literature

Several projects have documented research with multiple turn lanes, and this section summarizes findings.

Wortman (*48*) conducted a state-of-the-art review on double (then known as dual) left-turn lanes for the Arizona DOT in the 1980s. In it, he cited work from Neuman (*5*) that double left-turn lanes should be considered at any signalized intersection with high left-turn design-hour demand volumes and a general rule-of-thumb of 300 vehicles per hour or more as the appropriate demand volume for consideration of the double left-turn lanes. Other guidelines from Neuman (*5*) include the following:

- The throat width for the turning traffic is the most important design element. Drivers are most comfortable with extra space between the turning queues of traffic. Because of the offtracking characteristics of vehicles and the relative difficulty for acceptance of two abreast turns, a 36-ft throat width is desirable for acceptance of two lanes of turning

Table 5-1. Green Book material on double or triple left-turn lanes where additional research may be needed.

#	2011 Green Book Statements	Potential Additional Research Needs	Potential Source for Information
1	"…with three-phase signal control, such an arrangement results in an increase in capacity of approximately 180% of that of a single median lane."	Does the 180% value need to be updated/verified? Is the saturation flow rate affected by the location of a downstream friction point such as a bus stop or driveway?	Data are available from previous studies regarding saturation flow rates for triple and double left-turn lanes. These studies; however, are not sensitive to factors of interest such as turn lane width, receiving leg width, and downstream friction points. A study by Sando and Moses (*74*) identified some geometric features that influence saturation flow rate for triple left turns. Additional study that considers left-turn lane width, downstream friction points, and receiving leg width is needed.
2	"Occasionally, the two-abreast turning maneuvers may lead to sideswipe crashes."	Is more current information available? What design characteristics affect the crash rate (e.g., lane width on approach, receiving lane/leg width, presence of pavement markings, location of downstream friction point(s))	Study needed.
3	"The receiving leg of the intersection should have adequate width to accommodate two lanes of turning traffic. A width of 9 m [30 ft] is used by several highway agencies."	Is more current information available?	Study needed.
4	"Triple left-turn lanes have also been used at locations with very high left-turn volumes."	Should more insight into triple left-turn lane operations be provided?	Material to add to Green Book may be available in the recent TxDOT 0-6112 project (*52*), the FHWA *Signalized Intersections: Informational Guide* (*7*), and a report from a Florida study (*73*).
5	"…longitudinal lane line markings of double or triple left lanes may be extended through the intersection area to provide positive guidance."	Should this statement be followed by a reference to the MUTCD?	Reference could be added to MUTCD Section 3B.08 (Extensions Through Intersections or Interchanges.

traffic. In constrained situations, 30-ft throat widths are acceptable minimums.

- Guiding pavement markings to separate the turning lanes are recommended. These channelization lines should be carefully laid out to reflect offtracking and driving characteristics.
- Double left-turn lanes operate at approximately 1.8 times the capacity of a single left-turn lane.

Wortman (*48*) also cited some studies showing that the capacity of either lane in a double left-turn lane approach was lower than a single left-turn lane (e.g., a lane in a double left-turn might have 80% of the capacity of a single left-turn lane) and that the inside (median-side) left-turn lane had a somewhat lower capacity than the outside (shoulder-side) left-turn lane.

Ackeret (*75*) sought a better understanding of the operational characteristics of single, double, and triple left-turn lanes through measurements of saturation flow rates. His study included 36 double left-turn lane approaches along with 23 single and 3 triple left-turn lanes within the Las Vegas metropolitan area. Evaluation of analysis plots for key intersection characteristics suggested the following: lane widths over 12 ft are associated with higher saturation flow rates, lower saturation flow rate is present in the absence of longitudinal lane line pavement markings that are extended through the intersection area, and concurrent left-turn movements and separation distances may influence left-turn saturation flow rates. His research found no significant difference in performance between the double left-turn inside lane and outside lane. He recommended a double "left-turn lane saturation flow rate on the order of 1870 pcphgpl" (*75*). Based on the suggested left-turn lane saturation flow rate and his findings for the neighboring through lane, he suggested that a higher left-turn factor on the order of 0.98 or 0.99 be used rather than the 0.95 value present in the 1994 *Highway Capacity Manual* (*76*). The 2010 *Highway Capacity Manual* (*8*) includes a left-turn adjustment factor of 1/1.05 or 0.952.

Brich (*49*) examined positive guidance pavement markings for double left-turn lanes in Virginia and concluded that the "prevailing opinion is that this type of pavement markings for double left-turn lanes facilitates safe and efficient movement through the intersection." A figure was provided to illustrate that in the cases where the receiving roadway has more than two lanes, the pavement markings should be installed to have the left-most turn lane enter the left-most travel lane on the receiving roadway.

NCHRP Report 505 (*51*) discusses characteristics of intersection geometry to accommodate trucks. The authors referenced the existing Green Book guideline that the desirable turning radius for a double left-turn lane is 90 ft, but they concluded that there was little else on the design of multiple turn lanes to account for turning trucks. They stated that the primary factor to consider in designing double left-turn lanes is vehicle offtracking or swept path width. When vehicles negotiate the turn side by side, the vehicles should not encroach on the adjacent travel lane. Because many factors affect the control turning radius of double left-turn lanes, they stated that it is necessary to provide guidance on the range of offtracking or swept path width of design vehicles for various turning radii. They determined offtracking and resultant swept path widths of several design vehicles for 90-degree turns with centerline turning radii of 50, 75, 100, and 150 ft using AutoTURN software. Based on their analysis, they recommended that an exhibit be included in the Green Book that indicates the swept path width of several design vehicles for centerline turning radii of 75, 100, and 150 ft, to provide flexibility in designing adequate turning paths for double left-turn lanes by allowing for interpolation of swept path widths for a range of turning radii.

Cooner et al. (*52*) conducted research in Texas on design and operations of triple left-turn and double right-turn lanes. The field studies in Texas collected both static (e.g., lane widths, grades, pavement markings, traffic signs, upstream and downstream conditions, and signal timing) and dynamic (e.g., volumes by lane, saturation flow rate, and critical events) data in order to evaluate design and operational performance. Researchers collected the data at five TLT and 20 DRT lane sites, primarily in the Dallas–Fort Worth and Houston urban areas. They reported the following as key findings for TLTL:

- Lane utilization patterns were varied for each of the five sites studied.
- All sites were T-intersections with peak-hour volumes from 646 to 2,846 vehicles.
- Lighted pavement markers used to delineate the lane lines between the TLTL were effective at reducing violations and well received by the public at one site.
- Saturation flow rates in Texas were consistent with earlier published national values.

Sando and Moses (*74*) and Sando and Mussa (*77*) examined the influence of intersection geometrics on the operation of triple left-turn lanes. Their studies identified several geometric characteristics that did and did not influence operations. Variables that did not influence saturation flow rate included intersection type (four-leg versus T-intersection), shadowing effect (shadow or no shadow), time of day (morning, afternoon, or evening), and lane (outermost, middle, or inner most). Variables that significantly influenced saturation flow rate included the following:

- Skewness (skewed intersections resulting in less than 90-degree left turns had higher saturation flow rates than right-angled intersections).
- Street type (two-way streets had higher saturation flow rate than one-way streets, which was attributed to the tightness of the turning curve for the left-turning vehicles on the one-way streets).
- Approach grade (downgrade had higher saturation flow rates than level grade).
- Approach type (straight approaches had higher saturation flow rates than curved approaches).

They noted that there is a need to investigate downstream attraction bias. The left-turn lane width was provided but not discussed, and receiving lane width was not discussed in these studies.

Another variable known to affect left-turn saturation flow rate is U-turns. A 2005 study (*78*) measured vehicle headways in exclusive left-turn lanes at 14 signalized intersections—six sites had single left-turn lanes and eight had double left-turn lanes where only the inside turn lane was studied (because that was the lane assumed to be affected by U-turns). Regression analysis of saturation flow rate data showed a 1.8% saturation flow rate loss in the left-turn lane for every 10% increase in U-turn percentage. The safety analysis in that study used a set of 78 intersections. One of the findings was that sites with double left-turn lanes were found to have a significantly greater number of U-turn collisions.

The 2010 version of the *Highway Capacity Manual* (*8*) provides a lane width adjustment factor to "account for the negative impact of narrow lanes on saturation flow rate and allows for an increased flow rate on wide lanes." The values provided are 0.96 (average lane width of 8 to <10 ft), 1.00 (average lane width between 10 and 12.9 ft), and 1.04 (average lane width of more than 12.9 ft). These values indicate that saturation flow rate is affected by lane width.

Objective and Measures of Effectiveness

The objective for this research study was to determine the effects of geometric characteristics on operations for double left-turn lanes. The following geometric variables were the focus of this study: receiving leg width, left-turn lane width, and downstream friction point (type and distance). The key measures of effectiveness for the field study were

- Saturation flow rate measured as number of vehicles per hour of green per lane.
- Lane utilization.

These measures were used to identify the geometric variables that influence operations at DLTLs.

Study Matrix

The initial task for the field study was to refine the study matrix based on comments from the panel, information available in the literature, and availability of sites with preferred site characteristics. The variables that might affect the operations were identified as being left-turn lane width, receiving leg width, and downstream friction point. To ensure sufficient variability in these study variables, the following ranges were used during site identification:

- Receiving leg width, measured at extension of stop bar:
  - Narrow, less than 26 ft.
  - Moderate, between 26 and 30 ft.
  - Wide, more than 30 ft.
- Left-turn lane width, average:
  - Less than 12 ft.
  - 12 ft or more.
- Location of friction point:
  - Near, within 150 ft of end of turn.
  - Medium, between 150 and 350 ft of end of turn.
  - Long, more than 350 ft from end of turn.

The left-turn lane width ranges were modified to have an 11.5-ft (rather than a 12-ft) breakpoint because of the limited number of sites identified with an average lane width of 12 ft or more.

A previous study (*74*) indicated that the following can affect results: presence of an intersection skew, downgrade on the approach, horizontal curve on the approach, and approach street being one-way or two-way. Therefore, the research team attempted to control for these variables by minimizing the number of sites with these characteristics. Table 5-2 lists the preferred site selection characteristics used during site identification.

Variables that may affect operation, but were not the focus of the study, needed to be controlled by being held at a constant level. For example, the approach grade and horizontal alignment can affect operations and safety; therefore, because this is known, this research focused on other variables. Sites with level grades and straight alignment were preferred.

Study Sites

The research team contacted colleagues to help identify double left-turn lane sites. Researchers also used aerial photographs in Google Earth to identify sites and to gather preliminary

Table 5-2. Preferred site selection characteristics.

Variable	Characteristics
Number of left-turn lanes	Two lanes (required)
Area type	Urban or suburban
Surface condition	Fair to good
Intersection sight distance	Adequate
Grade	Level (prefer between -2 and 2)
Main road number of lanes	Prefer four lanes
Cross road number of lanes	Four lanes (two receiving lanes for the left-turn movements)
Left-turn lane assignment	All left-turn lanes must be exclusive left-turn lanes
Left-turn lane length	Minimum of 140 ft
Main road horizontal alignment	Straight
Cross road horizontal alignment	Straight
Adjacent parking lane	Select sites without on-street parking
Street type	Prefer both streets to be two-way streets
Shadowed*	Prefer sites with shadowed turn lanes
Presence of intersection lane line extensions	Prefer sites with left-turn lane line markings extending into the intersection
Median	Prefer sites with raised median on both approaches
Posted speed limit	Prefer sites with 35, 40, or 45 mph posted speed limits
Skew	Prefer sites to be near 90 degrees

* In shadowed left-turn configuration, the median prevents the left-turn lanes from being accessed directly by upstream through lanes. In unshadowed configuration, a through lane terminates in a left-turn lane at the intersection. A partially shadowed condition exists when a through lane vehicle could enter a portion of the left-turn lane when maintaining a straight path.

characteristics of the sites. The goal was to collect data from sites that varied in the following characteristics:

- Left-turn lane widths (range between 9 and 12 or more ft).
- Receiving leg width (range between 24 ft and 40 or more ft).
- Distance along receiving leg to friction point (range between 10 ft and more than 450 ft).

These preliminary characteristics were used to select sites while also considering researchers' ability to efficiently collect data at these locations. More than 200 sites were considered during site selection.

Additional considerations during site selection included the following:

- The double left-turn bay needed to be a minimum of 140 ft so that a queue length of seven vehicles or more was possible; otherwise, saturation flow rate was not calculable.
- Researchers preferred a left-turn bay length of 200 ft, which could accommodate a queue length of 10 vehicles.

The goal was to collect data in a minimum of three states—data were collected in Texas (College Station, Bryan, and Houston), Arizona (Flagstaff, Phoenix, and Tucson), and California (San Leandro and Palo Alto). Obtaining the desired range of receiving leg width was the most difficult of the study variables, although finding sites with an average double left-turn lane width greater than 12 ft also proved challenging. Only two sites met the greater than-12-ft average lane width criterion. Several locations had the double left-turn lanes turning into a receiving leg that had three lanes. The presence of the third lane enabled drivers to adjust their travel path. Therefore, only sites with a two-lane receiving leg (or three lanes when the third lane is added from a channelized right-turn lane) were considered. All of the sites had longitudinal lane line markings extended through the intersection to provide positive guidance to the left-turning drivers.

Some of the sites had to be eliminated after site selection and initial data collection because an insufficient number of queues were recorded or the signal operated in both protected and permissive modes. Data reduction was completed for 26 sites.

The variables and their descriptions collected for each site used in the analysis are provided in Table 5-3. Table 5-4 and 5-5 list the study site characteristics for the double left-turn lane approach and the receiving leg, respectively.

Table 5-3. Descriptions for the site variables.

Variable	Description
Site	Site name
A-PSL	DLTL approach: posted speed limit (mph)
A-LT_W	DLTL approach: average width of left-turn lanes (ft)
A-Med_W	DLTL approach: median width (ft)
A-Dis_Sig	DLTL approach: distance to nearest signal (ft)
A-Sig0-5	DLTL approach: number of signals within 0.5 mile
A-DwRt	DLTL approach: number of driveways within 1000 ft on right edge
A-DwLf	DLTL approach: number of median openings/driveways within 1000 ft on left edge
A-Bar-Nose	DLTL approach: stop bar to median nose (ft)
A-DLTL_Len	DLTL approach: length of DLTL, a minimum value was required to include as a study site (ft)
A-Tap_Len	DLTL approach: length of the taper (ft)
A-Tap_Des	DLTL approach: type of taper design; either curved, straight, or none
A-Shadow	DLTL approach: type of shadow design; either shadow, partial shadow, or two-way, left-turn lane
A-Bar-Curb	DLTL approach: distance between stop bar and extension of curb line (ft)
Ri	Turn: Radius at start of turn (ft)
Rf	Turn: Radius at end of turn (ft)
R-Lg_W-bar	Receiving leg: width of the receiving leg at extension of stop bar (ft)
R-Lg_W-100	Receiving leg: width of the receiving leg 100 ft beyond extension of stop bar (ft)
R-TH_W-100	Receiving leg: average width of the through lanes on receiving leg, 100 ft beyond extension of stop bar (ft)
R-Bike	Receiving leg: presence of bike lane (yes/no)
R-Dis_Sig	Receiving leg: distance to downstream signal (ft)
R-Sig0-5	Receiving leg: number of signals within 0.5 mile
R-DwRt	Receiving leg: number of driveways within 1000 ft on right edge
R-DwLf	Receiving leg: number of median openings/driveways within 1000 ft on left edge
R-Friction	Receiving leg: type of friction point, either bus stop, channelized right-turn lane, driveway/intersection, or friction is >500 ft downstream
R-D_Beg_Fric	Receiving leg: distance to start of friction point (499 used when >500 ft) (ft)
R-SP_Add100	Receiving leg: lane added on the receiving leg within 100 ft of intersection (yes/no)

Table 5-4. Site characteristics on double left-turn lane approach.

Site	A-PSL	A-LT_W	A-Med_W	A-Dis_Sig	A-Sig0.5	A-DwRt	A-DwLf	A-Bar-Nose	A-DLTL_Len	A-Tap_Len	A-Tap_Des	A-Shadow*	A-Bar-Curb
AZ-FS-03	35	12.0	3	1382	1	4	0	7	438	101	curved	SR	15
AZ-FS-04	35	12.0	4	455	1	1	1	2	150	94	curved	PS	23
AZ-FS-05	35	12.0	5	1485	1	0	0	4	315	88	curved	PS	25
AZ-FS-06	40	12.0	1	3000	0	0	1	3	225	N/A	straight	TW	27
AZ-FS-07	40	11.0	1	1097	1	6	0	0	218	N/A	straight	TW	42
AZ-PH-02	40	10.0	4	885	3	2	1	-5	170	120	curved	SR	19
AZ-PH-06	30	11.0	6	800	2	3	6	10	200	136	curved	SR	25
AZ-PH-07	40	12.0	1	2700	0	1	1	0	268	N/A	straight	TW	25
AZ-PH-08	35	9.5	1	553	2	8	4	5	150	183	curved	SR	22
AZ-PH-09	40	10.5	4	355	2	2	2	0	260	N/A	none	SR	48
AZ-PH-12	45	10.5	4	363	3	3	2	3	271	N/A	none	SR	50
AZ-PH-13	45	10.0	3	367	2	2	2	2	265	N/A	none	SR	59
AZ-PH-15	45	11.5	4	368	2	6	7	1	271	N/A	none	SR	53
AZ-PH-16	45	10.5	3	364	3	1	1	0	266	N/A	none	SR	55
AZ-TU-09	45	12.0	3	2810	1	4	2	9	250	425	curved	SR	29
AZ-TU-10	45	13.0	6	5000	0	2	3	14	410	300	curved	SR	45
CA-BA-04	35	11.0	4	446	3	10	1	0	205	214	curved	SR	21
CA-ST-01	35	11.0	4	1266	2	5	3	-6	344	113	curved	SR	23
CA-ST-02	35	10.5	4	1200	3	6	4	-5	284	128	curved	SR	28
CA-ST-04	35	11.0	5	1336	2	0	3	4	280	112	curved	SR	46
TX-CS-01	40	11.5	3.5	1900	1	3	1	7.5	140	121	curved	SR	32
TX-CS-02	40	13.0	3	1900	1	6	0	5	309	137	straight	SR	10
TX-CS-03	40	11.0	3	2635	1	4	5	4.5	151	148	curved	SR	20
TX-CS-04	45	12.0	2.5	1219	1	0	0	0	422	223	straight	SR	20
TX-HO-02	35	9.5	1.5	1584	1	4	2	3	124	121	curved	SR	18
TX-HO-03	45	12.0	7	975	1	4	0	5	223	150	curved	SR	27

*A-Shadow, SR = shadow with raised median (no direct entry to turning lanes, i.e., driver must change lanes to enter turn lane), PS = partial shadow, TW = two-way, left-turn on approach.

Figure 5-1 depicts the site characteristics gathered by technicians for each site. The descriptions of the per-queue variables are provided in Table 5-6.

Data Collection

The data collection method used was video recording. Video provided a permanent recording of the conditions at the site. Prior to beginning data collection, data collectors visited the site to determine the best locations to set up equipment, and they coordinated as needed with local authorities and adjacent landowners. The length of recording at each site was generally between 3 and 6 hr, with a few sites being recorded across multiple days to ensure that a sufficient sample size was obtained.

Researchers typically used camcorders to collect data at the sites. These recorders created a time-stamped video recording of traffic conditions at each intersection during the study period. The video was used to obtain the time each double left-turning vehicle departed the stop bar. These times were used to calculate the headway data and, from that data, the saturation flow rate. The video also provided the opportunity to validate conditions (e.g., presence of pedestrians or if the driver made a U-turn maneuver). The recording from another camera provided a record of lane changing just downstream of the turn due to the presence of a friction point (e.g., bus stop or driveway), any erratic maneuvers (e.g., panic braking, swerving), or conflicts.

Typically, a three-camera setup was used at each site, as shown in Figure 5-2. Camcorders were installed so that

- Camcorder #1 captured vehicles crossing the stop bar in the double left-turn lane. This view was the view of highest importance for the study because the wheels crossing the stop bars must be easily seen in the video. The view of Camera 1 had to be wide enough (but not too wide) to catch at least the first two vehicles in the queue (before the stop bar) and also the vehicles crossing the stop bar and entering the intersection.
- Camcorder #2 captured the entire double left-turn lane queue, especially vehicles 7, 8, 9, and 10 in the queue.

Table 5-5. Site characteristics on receiving leg.

Site	Ri	Rf	R-Lg_W@bar	R-Lg_W@100	R-TH_W@100	R-Bike	R-Dis_Sig	R-#Sig0.5	R-#DwRt	R-#DwLf	R-Friction*	R-D_Beg_Fric	R-Sp_Add100
AZ-FS-03	107	78	29	42	13	no	1030	1	0	0	RTL	33	Yes
AZ-FS-04	67	94	31	25	12	no	575	1	2	1	D/I	234	No
AZ-FS-05	85	101	46	29	11	yes	1605	1	1	1	>500	999	No
AZ-FS-06	58	97	34	31	11	yes	1056	1	1	0	D/I	289	No
AZ-FS-07	77	109	33	30	12	yes	1056	1	0	1	>500	999	No
AZ-PH-02	80	82	25	23	11	no	>0.5	0	1	1	D/I	400	No
AZ-PH-06	72	96	26	25	11	no	695	2	1	3	BS	110	No
AZ-PH-07	62	83	42	37	12	no	2600	1	8	2	BS	30	Yes
AZ-PH-08	67	84	25	24	10.5	yes	773	3	3	5	BS	57	No
AZ-PH-09	94	65	46	24	12	no	>0.5	1	0	0	>500	999	No
AZ-PH-12	93	69	48	24	12	no	>0.5	1	0	0	>500	999	No
AZ-PH-13	85	76	52	24	12	no	>0.5	1	0	0	>500	999	No
AZ-PH-15	88	71	52	24	12	no	>0.5	1	0	0	>500	999	No
AZ-PH-16	88	77	54	24	12	no	>0.5	1	0	0	>500	999	No
AZ-TU-09	93	111	48	35	12	yes	>0.5	1	3	5	D/I	365	No
AZ-TU-10	128	135	28	27	12	yes	1839	1	3	0	D/I	135	No
CA-BA-04	80	86	26	25	11.5	no	466	3	5	4	D/I	90	No
CA-ST-01	78	95	34	34	11.5	yes	>0.5	1	10	3	D/I	130	No
CA-ST-02	82	101	35	35	11.5	yes	734	2	3	4	BS	150	Yes
CA-ST-04	83	129	31	28	11.5	yes	>0.5	1	0	0	RTL	42	No
TX-CS-01	82	92	24	32	12	yes	1970	1	0	1	RTL	14	No
TX-CS-02	75	94	36	38	12	no	1130	1	3	2	D/I	414	No
TX-CS-03	83	97	25	37	11	no	1115	2	0	1	RTL	13	Yes
TX-CS-04	60	88	40	41	12	no	>0.5	0	1	1	RTL	20	Yes
TX-HO-02	73	102	30	21	12	no	2100	1	4	2	D/I	64	No
TX-HO-03	103	100	31	23	11	no	>0.5	0	1	2	D/I	300	No

*R-Friction: RLT = channelized right-turn lane, D/I = driveway/intersection, BS = bus stop, >500 = friction point is more than 500 ft from intersection.

- Camcorder #3 captured vehicles turning in the intersection and erratic vehicle maneuvers caused by the friction location. Camcorder #3 was placed about 20 ft after the friction location on the receiving leg (see Figure 5-3) in order to have a good view of the left-turn vehicles traveling from the intersection toward the camera and passing the friction location. If there was no friction location within 450 ft of the intersection (on the receiving leg), then Camcorder #3 was mounted approximately 450 ft downstream of the intersection (on the receiving leg) where there was a good view of all the vehicles traveling from the intersection toward the camera.

The research team also collected roadway geometry data for each study site. Typically, the geometric data were obtained using aerial photographs. These measurements were confirmed in the field or the geometric data were recorded on pre-printed worksheets created for this study.

Data Reduction

Saturation Flow Rate

Technicians reviewed the video and documented the time each left-turning vehicle crossed the stop bar. These times were used to determine the headway between following vehicles. Technicians also recorded whether the vehicle was a truck or whether the vehicle was in the queue at the start of the cycle. If either case was true, then the queue was eliminated from the study. Both Camera 1 and Camera 2 views were used to gather the vehicle type data and the last vehicle in queue data.

According to the *Highway Capacity Manual* (*8*) procedure, each cycle must have more than eight vehicles to be considered in the determination of saturation flow rate. The ITE *Manual of Transportation Engineering Studies* (*79*) recommends using the seventh, eighth, ninth, and tenth vehicle in the queue. Both procedures require the first four vehicles to be dropped in the analysis to eliminate headways with startup lost times and that only the passenger cars in the traffic stream are to be included. During early stages of data

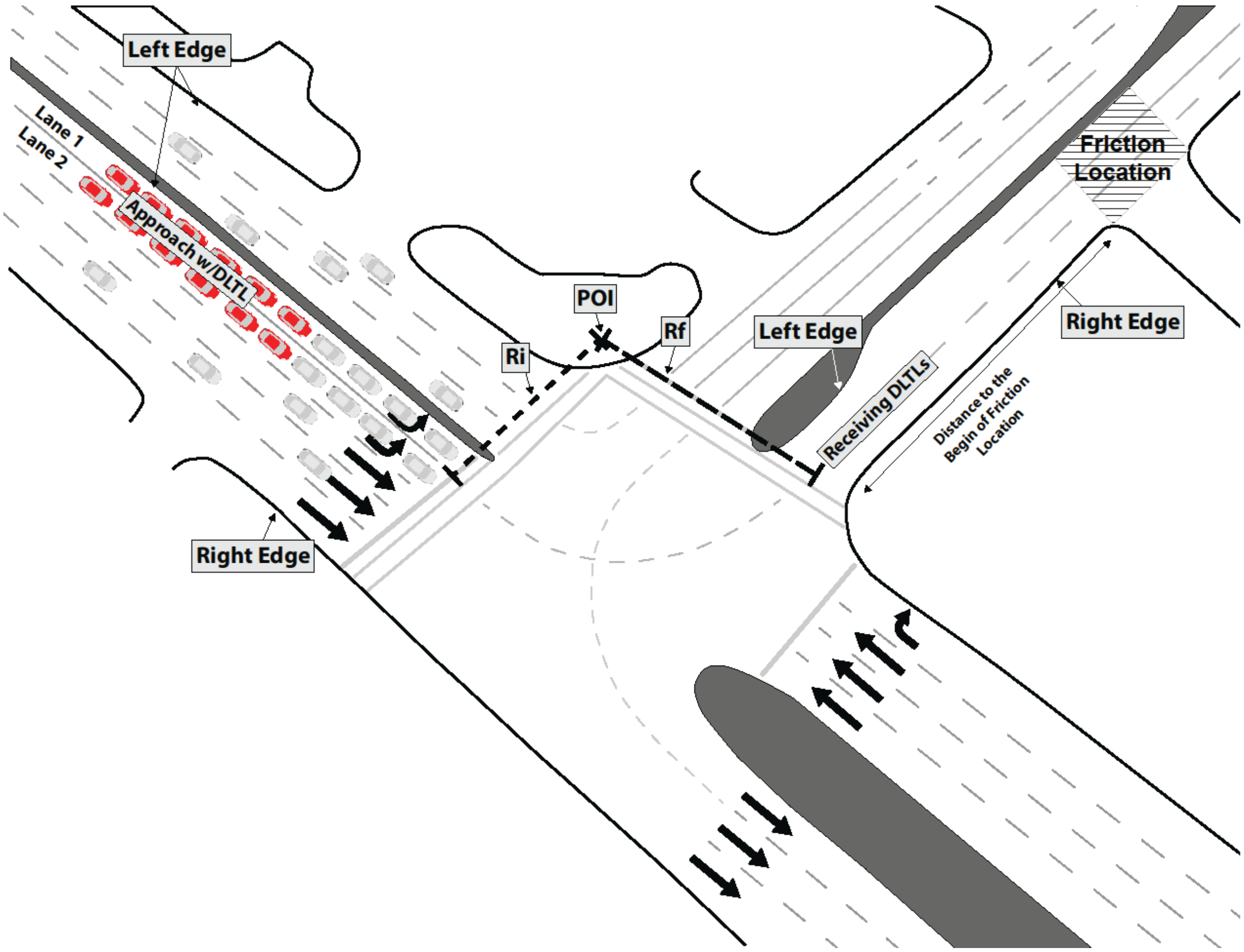


Figure 5-1. Graphic used to assist with gathering site characteristics.

reduction, few sites had sufficient number vehicles within the queue. The research team directed the field crew to record for 4 hr, or longer, to record at least 35 queues of eight passenger cars. Even with the longer recording times, several sites had less than 35 queues of eight passenger cars. Because this study focused on how geometric design variables (e.g., lane width) affected operations, the research team decided to retain the data for queues that had fewer than eight vehicles. The number of vehicles in the queue was included in the analysis to control for the effects that the queue length may have had on operations, and passenger cars in positions 4 through 10 of a queue were used in this study.

Saturation flow rate was calculated for each passenger car using the following equation:

$$SFR = 3600/(H/VQ\text{-}4)$$

where

SFR = saturation flow rate in passenger cars per hour of green per lane (pcphgpl).

H = time headway between subject vehicle and the fourth vehicle in the queue (sec).

VQ = subject vehicle position in the queue (e.g., 5, 6, 7, 8, 9, or 10).

Table 5-6. Descriptions for the per-queue variables.

Variable	Description
SFR or SatFlowRate	Calculated saturation flow rate using the average headway, in passenger cars per hour of green per lane (pcphgpl)
Site	Site name
Lane	Lane number, where 1 = inside lane and 2 = outside lane
Vehicle Queue	Number of vehicles within the queue used to calculate the saturation flow rate
Same Queue	The same value in this column (for a given site) indicates that the SFR was based on vehicles used in another SFR calculation
U-turns-w/in-queue	The number of vehicles within a given queue that performed a U-turn

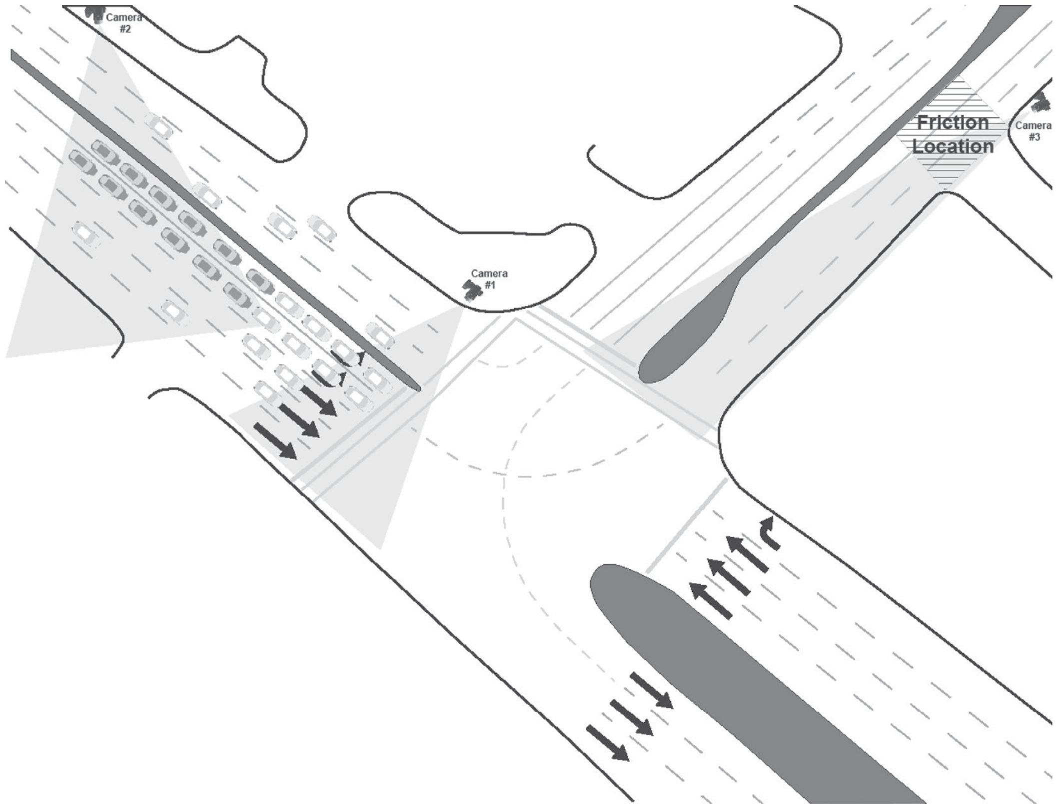


Figure 5-2. Example of video camera setup for double left-turn lanes study.

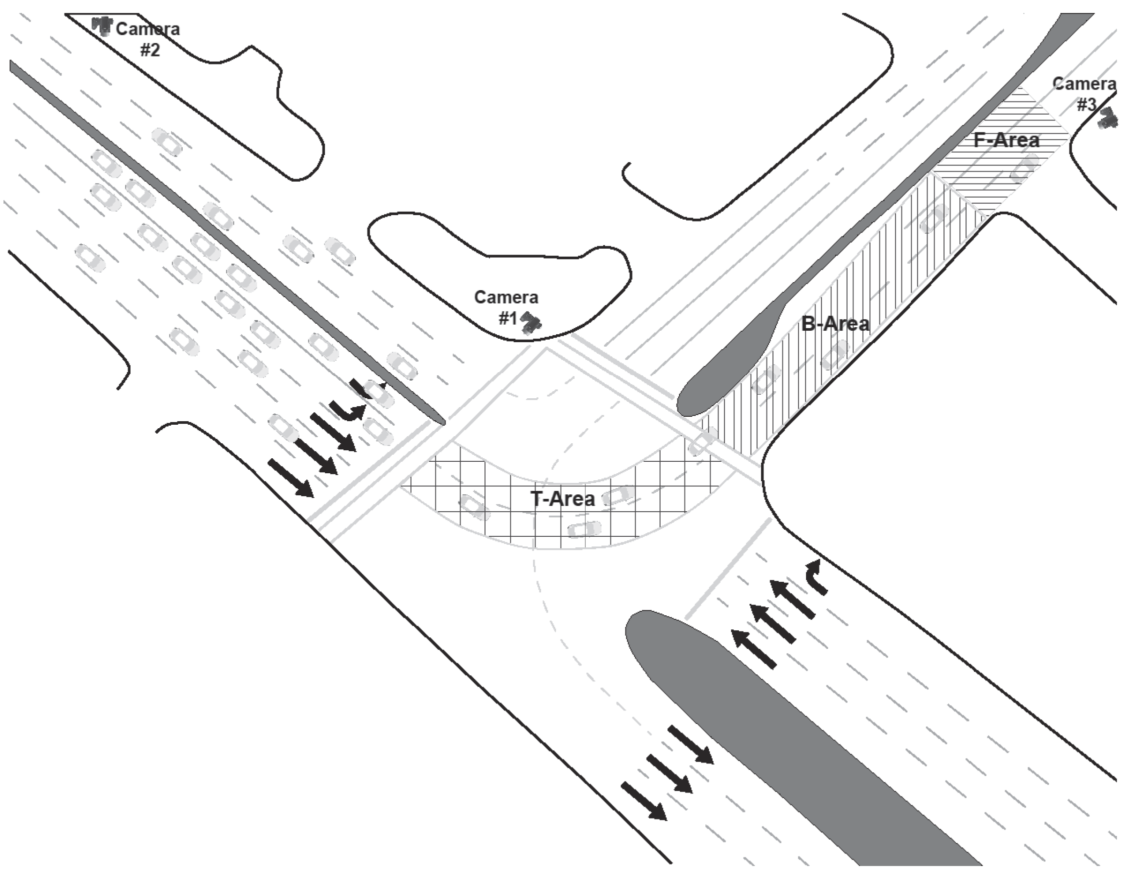


Figure 5-3. Subareas used while reducing Camera 3 driver behavior.

To handle extreme outliers, researchers decided to use a maximum selected saturation-flow-rate value based on an assumed headway of 1 sec, which resulted in a maximum saturation flow rate of 3600 pcphgpl value. A total of 18 of 10,041 data points were removed (less than 0.2%), resulting in 10,023 data points available for analysis. While a smaller saturation-flow-rate value could be used as the maximum value, the research team decided to start with a value that would eliminate obvious data reduction errors. Table 5-7 lists the average saturation flow rates by site for each lane.

Lane Distribution

At some DLTL intersections, drivers may choose one left-turn lane over another left-turn lane in anticipation of a turn at a downstream intersection or because of familiarity with a downstream friction point. The data available in this study were used to investigate if geometric elements are associated with how drivers distribute within the double left-turn lanes.

The *Highway Capacity Manual* (*8*) has a lane utilization factor to account for the unequal distribution of traffic among the lanes in those movement groups with more than one exclusive lane. A lane utilization factor of 1.0 indicates traffic is evenly distributed across different lanes in the movement group. Values less than 1.0 indicate traffic is not evenly distributed. The lane utilization factor, f_{LU}, can be computed using the following:

$$f_{LU} = V_g/(V_{g1}N)$$

where

f_{LU} = lane utilization factor.
V_g = unadjusted demand flow for the lane group (veh/hr).
V_{g1} = unadjusted demand flow on a single lane in the lane group with the highest volume (veh/hr).
N = number of lanes in the lane group.

Unadjusted demand flow refers to traffic flow not adjusted to take into consideration uneven traffic distribution within a lane group. Because of the nature of the equation, which lane has the higher volume is not associated with the factor. Therefore, another measure was needed in this study so that the proportion of vehicles in a given lane can be compared to the conditions at the site.

Table 5-7. Average saturation flow rate (pcphgpl) for each lane and site.

Site	SFR Average Lane 1	Count Lane 1	SFR Average Lane 2	Count Lane 2	SFR Average Both Lanes	Count for Site
AZ-FS-03	1777	147	1839	164	1810	311
AZ-FS-04	1611	172	1629	180	1620	352
AZ-FS-05	1688	226	1730	280	1711	506
AZ-FS-06	1630	191	1645	498	1641	689
AZ-FS-07	1905	1	1776	39	1779	40
AZ-PH-02	1607	72	1782	140	1722	212
AZ-PH-06	1864	36	1798	38	1830	74
AZ-PH-07	1972	175	1858	178	1915	353
AZ-PH-08	1789	18	1818	30	1807	48
AZ-PH-09	1633	17	1791	6	1674	23
AZ-PH-12	1757	343	1749	132	1755	475
AZ-PH-13	1846	362	1866	258	1854	620
AZ-PH-15	1783	381	1771	283	1778	664
AZ-PH-16	1845	315	1799	289	1823	604
AZ-TU-09	1931	450	1888	394	1911	844
AZ-TU-10	1699	423	1680	322	1690	745
CA-BA-04	1942	3	1721	14	1760	17
CA-ST-01	1735	339	1655	366	1693	705
CA-ST-02	1820	235	1766	268	1792	503
CA-ST-04	1783	58	1769	44	1777	102
TX-CS-01	1647	101	1725	99	1685	200
TX-CS-02	1652	136	1741	131	1695	267
TX-CS-03	1780	144	1845	122	1810	266
TX-CS-04	1842	244	1954	296	1903	540
TX-HO-02	1754	319	1860	421	1814	740
TX-HO-03	1631	84	1757	39	1671	123
Total	**1774**	**4992**	**1776**	**5031**	**1775**	**10023**

Table 5-8. Driver behavior recorded from Camera 1.

Code	Description
U-Turn	Did the vehicle make a U-turn?
Over Bar	Did the vehicle stop beyond the stop bar? A suggested criterion is, was any part of the vehicle's tire on the stop bar?
Backing Up	Did the vehicle back up?
Pedestrian/Bike Conflict	Was there a conflict between a pedestrian or bicyclist and the left-turning vehicle?
Slow to Start	Was the driver distracted and slow to start the turn? A suggested criterion is, was the vehicle more than one car length behind vehicles in the neighboring lane?

A variable was created to calculate the percent of the volume present within a cycle to each lane. This lane share variable is aimed at determining the proportion of vehicles that use a lane out of all left-turn vehicles recorded during a cycle. It is to provide a good measure of the distribution of left-turn demand of passenger cars across the double left-turn lanes. The lane share variable, L-share, was calculated as

$$L_i\text{-share} = V_i/(V_t/N)$$

where

L_i-share = proportion of vehicles in lane i relative to total left-turn volume.

V_i = volume for lane i (veh/hr).

V_t = total volume for both lanes.

N = number of lanes in the lane group, 2 for double left-turn lanes.

A previous study on triple left-turn lanes (*74*) found that some intersections had higher inside lane usage factors than the outer lane while other intersections were observed to have the opposite results. The authors made the following observations from their data:

- One intersection with an on-street bus stop less than 500 ft after the left turn had lower lane usage of the outer lane.
- Innermost lanes of the shadowed intersections were less used while the innermost lanes of the unshadowed intersections were highly used.

They noted that drivers' choice of lane results from many factors, including upstream and downstream conditions; however, they were not able to quantify the relationships.

Driver Behaviors

Driver behavior may be related to the geometric design characteristics of the double left-turn lanes, which could affect the saturation flow rate (i.e., operations) or the safety of the intersection. For example, slow-to-start behaviors may decrease the saturation flow rate and increase the potential for rear-end crashes.

The video recordings from Cameras 1 and 3 were used to identify driver behaviors of interest. Camera 1 was used to record driver behaviors near the stop bar. Table 5-8 lists the definitions of each behavior recorded using Camera 1.

Camera 3 was used to record driver behaviors after the drivers passed the stop bar. The Camera 3 view was subdivided into three areas (see Figure 5-3): T-area (after turning left and entering the cross street), B-area (after crosswalk, before the friction location), and F-area (at the friction point). The main function of Camera 3 was to record the movements of left-turn vehicles within the turn and near the friction point along the receiving leg. The driver behaviors listed in Table 5-9 were noted from watching the Camera 3 view. Driver behavior data were reduced for approximately 2 hr of video for each site that had a friction point within 450 ft of the intersection and that had an acceptable view of the left-turning vehicles.

Table 5-9. Driver behavior recorded from Camera 3.

Area	Code	Description
T	RT	Conflict with Right-Turning Traffic (Opposing Traffic)
	LT	Conflict with opposing Left-Turning Traffic
	PT	Conflict with Ped/Bike at the Crosswalk of the Cross Street
	DT	Stopping/Sudden Deceleration after Turning Left
B	BT	Lane Change before fiction location
	IB	Left-Turn into Improper Lane between the Intersection & Friction Location
	SB	Low Speed between the Intersection & Friction Location
F	DF	Stopping/Sudden Deceleration at Friction Location
	CF	Lane Change at Friction Location
	PF	Conflict with Ped/Bike/Traffic Exiting or Entering the Driveway (Back of Queue)
	DQ	Stopping/Sudden Deceleration after turning left due to Queue downstream on cross street

Analysis/Results

Saturation Flow Rate

Saturation flow rate represents the number of vehicles served by one lane over 1 hr of green time. It is calculated using the headway between following vehicles when all vehicles being considered were passenger cars and were present at the start of the green phase. The headways for the first four vehicles are dropped from the calculation.

The saturation flow rate data were analyzed using the analysis of covariance (ANACOVA) mixed model including several site variables in Table 5-10 as well as the number of vehicles within the queue used to calculate the saturation flow rate (vehicle queue) and the number of vehicles within a given queue that performed a U-turn (U-turns-w/in-queue) as fixed factors/covariates. This model considered each unique saturation flow-rate value at the intersections as opposed to averaging the saturation-flow-rate value for each intersection. Because individual saturation flow-rate values within the same queue from the same site are likely to be correlated, the variables of site and same queue were included as random factors to account for correlation. Parameters were estimated by the restricted maximum likelihood method implemented in JMP (SAS product).

Several models had a relatively good fit and provided insights into how the geometric features at the intersection affect the double left-turn lanes operations. The model selected as the most informative is shown in Table 5-10. It includes two of the key variables of interest: lane width and receiving leg width. Distance to friction point variable was removed from the model for several reasons, as will be discussed below. To verify that the results would be similar without the non-significant variables, another run was made using only the significant variables (see Table 5-11).

Lane

A 1987 study (*48*) stated that the inside turn lane had lower capacity than the outside left-turn lane, a finding not supported by more recent studies (*74, 75, 77*). The results from this dataset of 26 sites also support the finding that there are no differences in performance between the two double left-turn lanes. The results in Table 5-10 for the Lane[1] variable demonstrate that there is no significant difference in saturation flow rate between the inside and the outside lane.

U-Turns

As part of the data reduction efforts, whether the turning vehicle made a U-turn rather than a left turn was recorded. The number of U-turns made within a cycle was summed and considered within the analysis. Because U-turns require drivers to slow more than they would for a left turn, it is reasonable to assume that it will also take more time and therefore negatively affect saturation flow rate. The model found that for each additional U-turning vehicle within the queue, saturation flow rate would decrease by 56.45 pcphgpl. Stated in another manner, one U-turning vehicle is associated with

Table 5-10. Evaluation of saturation flow rate.

Response SFR

Summary of Fit

RSquare	0.697342
RSquare Adj	0.69716
Root Mean Square Error	229.6468
Mean of Response	1775.154
Observations (or Sum Wgts)	10023

Parameter Estimates

Term	Estimate	Std Error	DFDen	t Ratio	Prob>|t|
Intercept	1897.3516	169.7568	22.12	11.18	<.0001*
Lane[1]	1.918438	6.176225	2873	0.31	0.7561
Vehicle queue	-2.491214	1.753325	7535	-1.42	0.1554
U-turns-w/in-queue	-54.82995	10.50856	5650	-5.22	<.0001*
A-LT_W	-17.98789	14.09065	21.11	-1.28	0.2156
R-Lg_W-bar	3.2048467	1.413015	22.37	2.27	0.0333*
SP_Add100[No]	-52.31953	16.56282	21.42	-3.16	0.0047*

REML Variance Component Estimates

Random Effect	Var Ratio	Var Component	Std Error	95% Lower	95% Upper	Pct of Total
Site	0.0613196	3233.8501	1292.8548	699.90123	5767.7989	2.292
Same queue	1.6143177	85135.325	2911.7246	79428.449	90842.2	60.334
Residual		52737.65	906.7706	51004.58	54561.01	37.374
Total		141106.83	3150.3364	135130.27	147490.81	100.000

-2 LogLikelihood = 142648.28513

Note: Total is the sum of the positive variance components.

Total including negative estimates = 141106.83

Table 5-11. Evaluation of saturation flow rate using only significant variables.

Response SFR

Summary of Fit

RSquare	0.697047
RSquare Adj	0.696957
Root Mean Square Error	229.7227
Mean of Response	1775.154
Observations (or Sum Wgts)	10023

Parameter Estimates

Term	Estimate	Std Error	DFDen	t Ratio	Prob>|t|
Intercept	1679.5098	53.54089	24.08	31.37	<.0001*
U-turns-w/in-queue	-56.45924	10.13527	5069	-5.57	<.0001*
R-Lg_W-bar	3.1498985	1.423279	23.11	2.21	0.0370*
SP_Add100[No]	-49.65104	16.62346	22.51	-2.99	0.0067*

REML Variance Component Estimates

Random Effect	Var Ratio	Var Component	Std Error	95% Lower	95% Upper	Pct of Total
Site	0.0629358	3321.2783	1292.685	787.66217	5854.8944	2.354
Same Queue	1.6100791	84967.917	2905.5898	79273.066	90662.768	60.235
Residual		52772.51	907.13018	51038.741	54596.58	37.411
Total		141061.71	3144.6954	135095.57	147433.94	100.000

-2 LogLikelihood = 142667.5979

Note: Total is the sum of the positive variance components.

Total including negative estimates = 141061.71

a 3.4% decrease in saturation flow rate (–56/1680), while two U-turning vehicles are associated with a 6.7% decrease in saturation flow rate (–113/1680). The findings from this study show a larger impact of U-turning vehicles on saturation flow rate than the findings from a 2005 study (*78*).

Number of Vehicles in Queue

The number of vehicles in the queue was also not significant (see Table 5-10). In other words, whether the queue length was five vehicles or ten vehicles, similar saturation flow rates were measured after controlling for variations caused by other variables within the model.

Friction Point on Receiving Leg

The location and type of friction that would first be encountered by a left-turning driver was identified for each site. The type of friction was categorized as

- Channelized right-turn lane exit.
- Bus stop.
- Driveway or minor intersection.
- No friction within 450 ft of the intersection.

The distance between the friction point and the stop bar extension was measured. Because the friction points are actually not "points" but have a dimension to reflect the width of the driveway or the length of the bus stop, the distance was measured to the leading edge of the friction point. Analyses were conducted that considered the type of friction, the distance to the leading edge of the friction, and/or grouping the distances into reasonable ranges. One of the reasons the distances were grouped was to combine all the sites where the friction was more than 450 ft from the intersection. At several locations the next roadside friction was more than 1000 ft, which is beyond a reasonable distance that should be influencing the operations for the double left-turn lanes.

The type of friction was found to not be significant. Whether the friction was a driveway or a channelized right-turn lane exit did not influence the performance of the double left-turn lane operations. According to the modeling results, the distance to the leading edge of the friction did influence operations; however, it was in a manner not expected. As the distance to the friction increased, saturation flow rate decreased. Expected was that the closer a friction point is located to the intersection, the more operations of the double left-turn lane would be compromised. Additional evaluations into the site characteristics and the data revealed another variable that should be included in the model—a variable that reflects when a lane is added to the receiving leg. At several locations, the channelized right-turn lane added a downstream lane, in some cases within only a few feet of the intersection. While the turning vehicles were constrained at the start of the receiving leg, a review of the video data revealed that drivers in the outside lane would angle their vehicle to make a smooth entry into the new lane. This behavior resulted in higher saturation flow rates, as demonstrated with the variable SP_ADD100[No] being significant. The model results indicate that the addition of this new lane results in an increase in saturation flow rate of about 50 pcphgpl.

To determine the impacts of friction type and location would require a more detailed study that considers the type of friction (e.g., bus stop and driveway) and the specific action at the friction point (e.g., driver waiting on driveway, driver existing driveway, and bus stopped at bus stop) when the left-turning vehicle arrives. Also needed would be the action of the left-turning vehicle (e.g., turning into driveway or changing lanes to avoid activity at the friction point).

Left-Turn Lane Width

The width of the double left-turn lanes was thought to affect overall operations, especially as the *Highway Capacity Manual* (*8*) includes an adjustment factor for lane width. When lanes are wider, drivers may feel more comfortable and drive faster, which would be reflected in higher saturation flow rates. This analysis actually found the opposite with lower saturation flow rates with the wider left-turn lane widths; however, the result was not significant. Figure 5-4 shows the average saturation flow rate for each site along with the average for the left-turn lane width. While one can see a downward trend in saturation flow rate compared to left-turn lane width, the graph also shows large ranges of saturation flow rates. For example, sites with 12-ft lanes had both the highest and the lowest saturation flow rates. The graph indicates that more than just the left-turn lane width is affecting saturation flow rate. In summary, within this dataset, there is not enough evidence to suggest the width of the left-turn lanes at these sites influenced the saturation flow rate. The saturation flow rates used in this evaluation only included passenger cars. If a truck or a bus was present within the queue, the data were eliminated. A future study could investigate the effects of larger vehicles on double left-turn lane operations.

Receiving Leg Width

The width of the receiving leg was defined as the distance between the median and the curb. It represents the visual target for the left-turning drivers. A narrow receiving leg width may result in drivers turning more slowly as drivers have to take more care in positioning their vehicles to ensure that they do not hit the curb or the neighboring vehicle.

A consistent location for the measurement was needed, and the research team decided to use an imagined extension of the stop bar present for the opposing direction approach. When the receiving leg was a one-way road, the width of the receiving leg along the crosswalk marking was used for the measurement. Measuring the receiving leg width prior to the end of the corner radius makes it possible to account for the extra pavement available to turning drivers when a larger corner radius is present, when a tapered nose design is used for the raised median, or when, because of the angle of intersection, additional pavement is available to the turning driver.

Because the receiving leg width was one of the key study variables, sites were selected to represent a range of receiving leg widths. All sites had two lanes at the start of the receiving leg. A few sites had a third lane added to the leg from a channelized right turn; however; all of these sites had a raised channelized right turn. In other words, all turning drivers had to turn into only two lanes. Preliminary reviews of double left-turn lane operations revealed that when drivers can turn into three, rather than two, lanes, the turning behavior and, therefore, the saturation flow rate are affected. For the sites included in this study, the width of the receiving leg ranged from 24 ft to 54 ft. Figure 5-5 shows the average saturation flow rate per site by receiving leg width.

The Green Book states that "the receiving leg of the intersection should have adequate width to accommodate two lanes of turning traffic. A width of 9 m [30 ft] is used by several highway agencies." The analysis conducted as part of this study found that the width of the receiving leg affected the

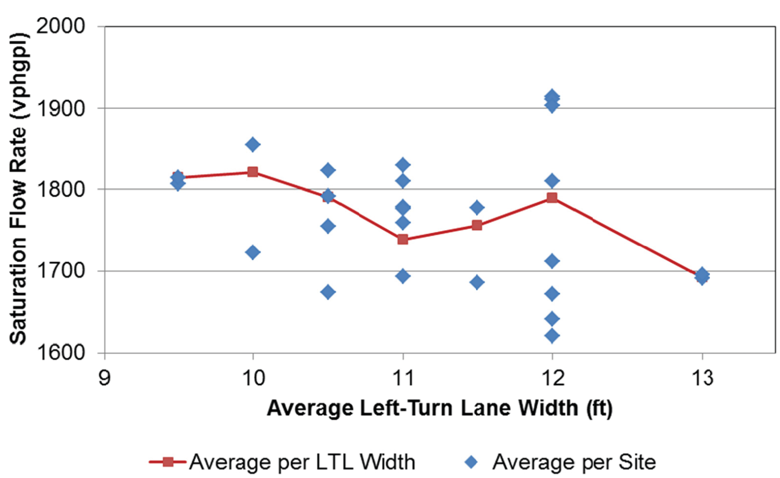


Figure 5-4. Saturation flow rate by left-turn lane width.

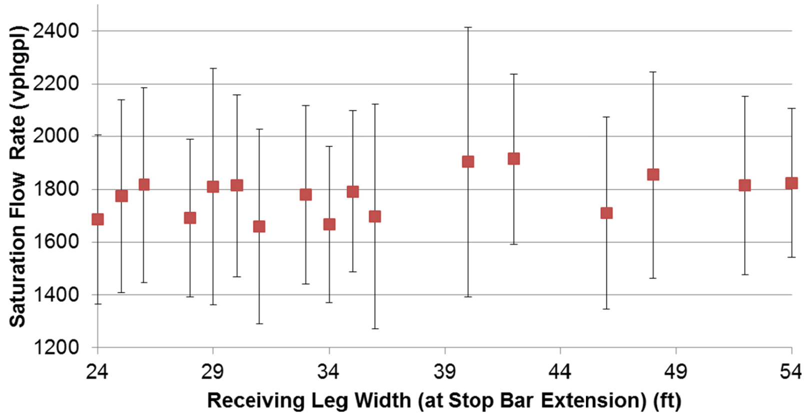


Figure 5-5. Saturation flow rate (site average with bars showing one standard deviation) by receiving leg width.

saturation flow rate of the double left-turning traffic. While significant, the incremental difference is small. Each additional foot of receiving leg width is predicted to increase saturation flow rate by only 3.2 pcphgpl (see Table 5-12). Within an analysis, both statistical and practical differences need to be considered. How to judge a practical difference is not as clearly defined as how to judge a statistical difference. One approach is to use the accuracy of the measurement technique (for example, 1 mph when measuring speed using radar). Another approach is to use a value that drivers could perceive, say 2 or 3 mph for a change in free-flow speed. A third approach could be to use a percentage of the average of the measurement. The average saturation flow rate for these sites is 1781 pcphgpl. If the variable contributes more than, say, 1% (18 pcphgpl) or 5% (89 pcphgpl), then one could argue that there is a practical difference. The receiving leg width variable represents a change of 3.2 pcphgpl for 1 ft of change. So a 5.6-ft change in the receiving leg width could be considered to have both a statistical and practical effect on saturation flow rate when using a 1% change in the average saturation flow rate.

The pattern of increasing saturation flow rate for increasing receiving leg width was examined to try to identify if there were dimensions where a sizable increase in saturation flow rate occurs. A change point detection method was used to detect a shift in the mean vector (and the covariance matrix) when the data set consists of multivariate individual observations (R-Lg_W-bar and average [SFR] for each site). The method concluded that the change point appears at R-Lg_W-bar=36. The average saturation flow rate by group is shown in Table 5-12.

Table 5-12. Average saturation flow rate by receiving leg width groups.

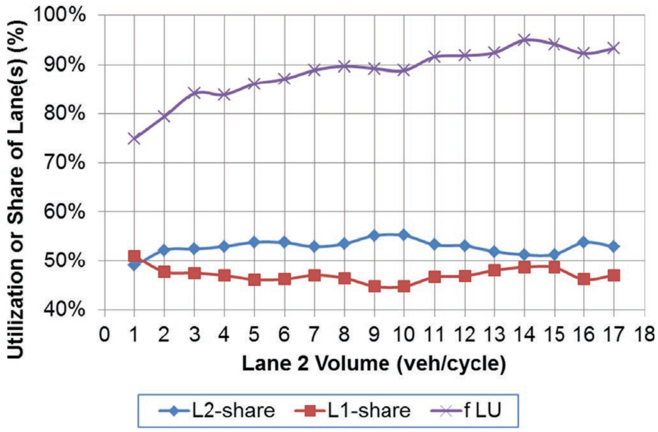
Receiving Leg Width (ft)	Average Saturation Flow Rate (pcphgpl)
24 to 36	1725
40 to 54	1833

These findings indicate that there is a difference in SFR when the receiving leg width is 24-36 ft and when it is 40 ft or more.

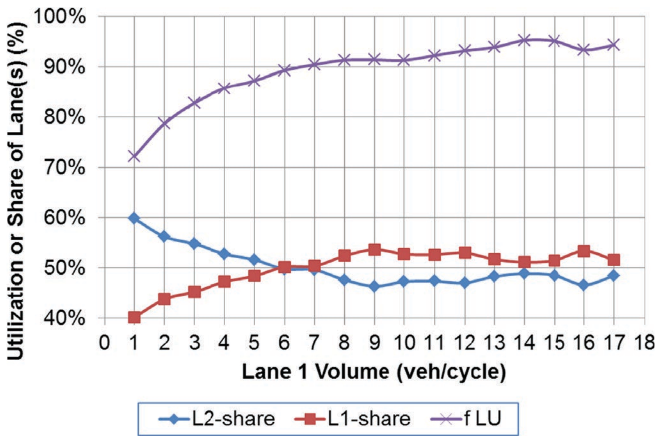
The sites are different not only in receiving leg width (R-Lg_W-bar) but also in other characteristics, so there could be some confounding if just the relationship between receiving leg width and saturation flow rate is examined without considering other factors. Therefore, an analysis of covariance was done based on the average SFR data to incorporate the effects of other variables in assessing the relationship between receiving leg width and average saturation flow rate. The least squares means for R-Lg_W-bar can be considered as the predicted saturation flow rates that have been adjusted for the effects of other factors in the model. The change point detection analysis based on those predicted saturation rates again identified receiving leg width of 36 ft as the change point.

Lane Distribution

As indicated earlier, the average lane utilization factor, f_{LU}, accounts for the unequal lane utilization of the double left-turn lanes as a lane group. Before investigating whether unique site characteristics were affecting lane distribution, the measures were examined to identify if traffic demands are also affecting lane distribution. As demand increases, the selection of which left-turn lane to enter may be more of a reflection of drivers selecting the lane with the shorter queue rather than being concerned with downstream conditions. The lane utilization factor along with the lane share factors were plotted using the volume in the outside lane (Lane 2 in Figure 5-6a) and the volume in the inside lane (Lane 1 in Figure 5-6b). As shown in both plots and as expected, there is a strong relationship between f_{LU} and the volume in Lane 1 or Lane 2. When more than approximately 7 vehicles for Lane 1 (see Figure 5-6b) or more than approximately 11 vehicles for Lane 2 (see Figure 5-6a) are present within a queue, the lane utilization factor is greater than 90%. Therefore, to be able to



(a) Lane Utilization by Lane 2 (Outside Lane) Volume



(b) Lane Utilization by Lane 1 (Inside Lane) Volume

Figure 5-6. Lane utilization by left-turn lane volume.

better focus on data when lane selection may be more influenced by geometric conditions, two datasets were created:

- Dataset 11 contained the data when 11 or fewer vehicles are in each lane.
- Dataset 7 included the cycles when at least one of the two lanes had seven or fewer vehicles.

A preliminary plot of the Lane 2 share results for Dataset 11 and Dataset 7 (see Figure 5-7) shows that the percent of vehicles in Lane 2 was less than 50 for some sites and more than 50 for other sites. The characteristics of the sites may be influencing which lane is being selected by drivers.

When using the dataset based on a maximum of 11 vehicles in a queue, the average f_{LU} was 90% (see Table 5-13). The overall L2 share average for the 26 sites was 0.51. In other words, the split between the inside and outside lanes was nearly even when averaging the data across all sites. When reviewing the findings per site (see Table 5-13), variations between sites are seen with a range of 0.44 to 0.58 for the L2 share variable.

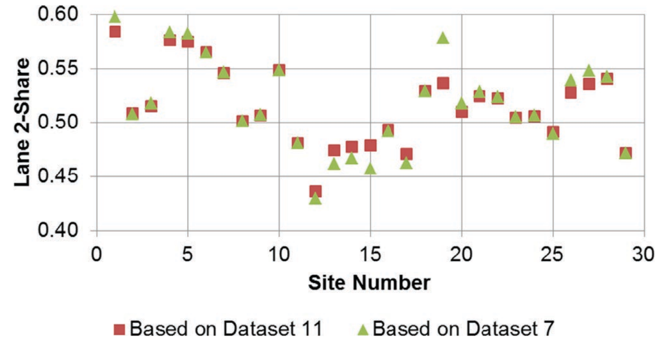


Figure 5-7. Lane 2 share data by site.

The L2 share data were analyzed using the ANACOVA mixed model, including several site variables in Table 5-3 as well as Lane 1 volume (L1_Vol) and Lane 2 volume (L2_Vol) as fixed factors/covariates. Because individual L2 share values from the same site may be correlated, the variable site was included as a random factor to account for potential correlation. Parameters were estimated by the restricted maximum likelihood method implemented in JMP. The results for the model selected as the best and most informative is shown in Table 5-14.

The variables found to be significant using Dataset 11 (within 0.10) included

- Lane 1 volume
- Lane 2 volume
- Median width on the approach
- Number of signals within 0.5 mi of the double left-turn lanes on the approach
- The radius at the end of the left turn
- The distance to the beginning of the friction point

Examining the results of the statistical evaluation reveals the following:

- Almost all of the L2 share prediction is accomplished by the Lane 1 and the Lane 2 volumes, indicating that drivers are making lane selection based on the length of queues present when the driver approaches the intersection.
- The expectation is that the proportion of drivers selecting Lane 2 would be smaller when the distance to the beginning of the friction point is small. In other words, drivers would avoid Lane 2 to evade potential conflicts at the nearby friction point. As expected, as the distance to the beginning of the friction point increases, the L2 share increases; however, even at the maximum distance available in the dataset (999 ft), the increase in L2 share is only 0.004. Therefore, practically speaking, the location of the friction point overall does not influence the decision of the driver regarding which lane to select within the double

Table 5-13. Average lane utilization factor and L2 share per site.

Site	Dataset 11			Dataset 7		
	f_{LU}	L2 share	Count	f_{LU}	L2 share	Count
AZ-FS-03	84%	0.58	76	81%	0.60	64
AZ-FS-04	88%	0.51	171	87%	0.51	146
AZ-FS-05	90%	0.52	92	88%	0.52	69
AZ-FS-06	86%	0.58	213	85%	0.58	192
AZ-FS-07	85%	0.57	23	85%	0.57	23
AZ-PH-02	87%	0.55	93	87%	0.55	92
AZ-PH-06	92%	0.50	64	92%	0.50	64
AZ-PH-07	93%	0.51	36	92%	0.51	25
AZ-PH-08	90%	0.55	35	90%	0.55	35
AZ-PH-09	93%	0.48	32	93%	0.48	32
AZ-PH-12	87%	0.44	127	86%	0.43	116
AZ-PH-13	93%	0.47	69	91%	0.46	25
AZ-PH-15	93%	0.48	101	90%	0.47	62
AZ-PH-16	95%	0.48	23	91%	0.46	7
AZ-TU-09	91%	0.49	81	91%	0.49	68
AZ-TU-10	91%	0.47	85	89%	0.46	58
CA-BA-04	92%	0.53	14	92%	0.53	14
CA-ST-01	91%	0.54	24	86%	0.58	10
CA-ST-02	91%	0.52	73	88%	0.53	52
CA-ST-04	91%	0.52	40	91%	0.52	37
TX-CS-01	91%	0.50	86	91%	0.51	82
TX-CS-02	89%	0.51	77	87%	0.51	63
TX-CS-03	93%	0.49	62	92%	0.49	54
TX-CS-04	92%	0.53	102	90%	0.54	69
TX-HO-02	90%	0.54	150	89%	0.55	115
TX-HO-03	89%	0.47	84	89%	0.47	83
Total	**90%**	**0.51**	**2033**	**88%**	**0.52**	**1657**

Table 5-14. Evaluation of L2 share.

Response L2 share

Summary of Fit

RSquare	0.948108
RSquare Adj	0.947955
Root Mean Square Error	0.019131
Mean of Response	0.513478
Observations (or Sum Wgts)	2033

Parameter Estimates

Term	Estimate	Std Error	DFDen	t Ratio	Prob>|t|
Intercept	0.5041318	0.006638	25.54	75.94	<.0001*
L1_Vol	-0.040686	0.00026	1521	-156.6	0.0000*
L2_Vol	0.0388758	0.000267	1753	145.56	0.0000*
A-Med_W	-0.002238	0.000557	21.63	-4.02	0.0006*
A-Sig0-5	0.0023339	0.001	24.41	2.33	0.0282*
Rf	0.0001337	0.000064	24.17	2.09	0.0473*
R-D_Beg_Fric	4.3374e-6	2.403e-6	24.19	1.80	0.0836

REML Variance Component Estimates

Random Effect	Var Ratio	Var Component	Std Error	95% Lower	95% Upper	Pct of Total
site	0.0291733	1.0677e-5	5.1257e-6	6.3056e-7	2.0723e-5	2.835
Residual		0.000366	1.1572e-5	0.0003443	0.0003898	97.165
Total		0.0003767				100.000

-2 LogLikelihood = −10184.46056

Fixed Effect Tests

Source	Nparm	DF	DFDen	F Ratio	Prob > F
L1_Vol	1	1	1521	24508.63	0.0000*
L2_Vol	1	1	1753	21187.57	0.0000*
A-Med_W	1	1	21.63	16.1674	0.0006*
A-Sig0-5	1	1	24.41	5.4448	0.0282*
Rf	1	1	24.17	4.3688	0.0473*
R-D_Beg_Fric	1	1	24.19	3.2577	0.0836

left-turn lane group. Note, however, that the dataset does not include information on whether there was activity at the friction point during a cycle. Considering the situation when activity is present may result in other conclusions.

- More drivers selected the outside lane at the sites with longer Rf and sites with a higher number of signals within ½ mi. Fewer drivers selected the outside lane at sites with wider medians. All of these differences are very small and not of practical value.

Driver Behaviors

The driver behaviors recorded with the camera used to determine saturation flow rate (i.e., Camera 1) included

- Did the vehicle make a U-turn?
- Did the vehicle stop beyond the stop bar?
- Did the vehicle back up?
- Was there a conflict between pedestrian or bicyclist and a turning vehicle?
- Was the driver distracted and slow to start the turn?

The number of vehicles within a queue making a U-turn was considered as part of the saturation-flow-rate evaluation. The saturation-flow-rate analyses clearly showed that the number of vehicles making a U-turn affects the operations at the signalized intersection (see Table 5-11).

For this dataset, no conflicts between a pedestrian or bicyclist and a left-turning vehicle were observed. A vehicle passing the stop bar and then backing up to be behind the stop bar was also not observed.

A few vehicles were observed stopping beyond the stop bar or being slow to start the turn. Only two of the 10,023 lead vehicles were observed stopping over the stop bar and less than 1% of the fifth to tenth vehicles in a queue were slow to start. Once the queue started to flow, very few left-turn drivers were distracted, in the opinion of the technicians reducing the data.

Driver behaviors after the turn had started and around the friction point had more interesting data. Figure 5-3 shows the subareas used when reducing driver behavior data from Camera 3. Table 5-15 summarizes the number of driver behaviors observed. For the 18 sites included in this review, the most common driver behavior recorded was lane changes before the friction location, followed by lane changes at the friction location.

Table 5-16 lists the sites by number of driver behaviors for those sites where more than 60 driver behaviors were observed. The three sites with the highest number of driver behaviors all had a driveway or intersection within 450 ft of

Table 5-15. Number of driver behaviors observed within 2 hr at each site.

Site	RT	LT	PT	DT	BT	IB	SB	DF	CF	PF	DQ	Total
AZ-FS-03					7	2			94	1		104
AZ-FS-04					64		7	16	18			105
AZ-FS-05					21							21
AZ-FS-06					43	1		4	9			57
AZ-PH-02					52	3						55
AZ-PH-06					6				13			19
AZ-PH-07						3			1			4
AZ-PH-08									8		18	26
AZ-TU-09				1	66		70	72	2			211
AZ-TU-10	1				11				7			19
CA-BA-04					7	1	17	3	5	2		35
CA-ST-01	2		1	7	2		14	13	7	3		49
CA-ST-02	5			8	4	1	1		9			28
CA-ST-04								1	1	1		3
TX-CS-02					109	1	1	9	8			128
TX-CS-03									46			46
TX-CS-04				4	1	5			34			44
TX-HO-03					6		5	2	4			17
Total	8	0	1	20	399	17	115	120	267	7	18	972

Note: RT = conflict with right-turning traffic (opposing traffic); LT = conflict with opposing left-turning traffic; PT = conflict with ped/bike at the crosswalk of the cross street; DT = stopping/sudden deceleration after turning left; BT = lane change before friction location; IB = left-turn into improper lane between the intersection & friction location; SB = low speed between the intersection & friction location; DF = stopping/sudden deceleration at friction location; CF = lane change at friction location; PF = conflict with ped/bike/traffic exiting or entering the driveway (back of queue); DQ = stopping/sudden deceleration after turning left due to queue downstream on cross street. Blank cells = 0.

Table 5-16. Sites with the highest number of driver behaviors.

Site	Total	Comments	R:Friction	Distance to Friction (ft)
AZ-TU-09	211	Many of the driver behaviors were right turns into the driveway. One vehicle could have multiple driver behaviors (BT, SB, DF).	Driveway/ Intersection	365
TX-CS-02	128	Most of the lane changes were into a left-turn lane for an entrance across from the friction point.	Driveway/ Intersection	414
AZ-FS-04	105	Similar to TX-CS-02, many lane changes into a left-turn lane that is near the friction point.	Driveway/ Intersection	234
AZ-FS-03	104	Lane changes into and out of the lane added by the right-turn channelization were most of the driver behaviors observed.	Channelized RTL	33

the intersection. Several other study sites also had a driveway or intersection near the intersection but did not have as many driver behaviors. While the presence of the downstream driveway or intersection can be associated with multiple drive behaviors of interest, it is not always the case. There were some lane changes before the friction point at all study sites that had a downstream driveway/intersection. This finding offers a caution that downstream driveways and intersections are associated with greater activity that can affect the operations of the roadway.

The addition of a lane on the major street from a cross-street right-turn lane is also associated with increased lane changing behaviors. Lane changes into and out of this additional lane were observed at several sites. While this additional lane is generally intended to provide a free-flow opportunity for the cross-street right-turning vehicles, at several of the study sites, left-turning drivers were observed to move directly into this lane, in some cases, over a solid white pavement marking.

The behaviors that reflected conflicts (RT, LT, PT, DT) along with potential conflicts—DT (stopping/sudden deceleration after turning left) and IB (left-turn into improper lane between the intersection and friction location)—were reviewed to identify if there were common elements. The three sites with the largest number of these types of behaviors—CA-ST-01, CA-ST-02, and TX-CS-04—all had a friction point within 150 ft of the intersection.

CHAPTER 6

Deceleration Field Study

Background

The objective of NCHRP Project 03-102 was to recommend improvements to the guidance provided in the AASHTO *Policy on Geometric Design of Highways and Streets* (commonly known as the Green Book) (*2*). Task 1 was to identify the state of design practice for auxiliary lanes at intersections by gathering and synthesizing information on existing practices and research. Task 2 determined issues that merit further study to validate, enhance, and expand current Green Book guidance. Task 3 produced the interim report and a Phase II work plan, which includes suggested research efforts for the remaining tasks. This chapter documents the method and findings for the deceleration study selected for Phase II of NCHRP Project 03-102.

Green Book Review on Deceleration (Section 9.7.2)

The 2011 Green Book discusses deceleration lanes in Section 9.7.2, and it does so in two ways: first by defining them and then describing what makes up each of their three component sections (i.e., deceleration length, storage length, and taper length). Table 6-1 lists potential research needs based on a review of Green Book material and related literature.

The definition section is new to the 2011 Green Book and also discusses a fourth component: distance traveled during perception-reaction (PR) time. Item 1 in Table 6-1 could be addressed by referencing one or more previous studies on the subject.

Item 2 deals with assumptions made on calculating appropriate deceleration distance. Section 9.7.2 of the 2011 Green Book states that a desirable objective on arterial streets is the provision for deceleration clear of the through-traffic lanes, and Table 9-22 in the same section provides estimates of the needed distances; however, the Green Book states that the lengths in the table should be accepted as a "desirable" goal and should be provided where "practical." The Green Book section presents justification for accepting a 10-mph speed differential between turning vehicles and through traffic on arterial roadways and states that higher speed differentials may be acceptable on collector highways and streets. The notes within Green Book Table 9-22 provide information that can be used to generate the values, but it was not known whether the 10-mph differential was realistic for current drivers or whether the other assumptions made in the calculations needed to be updated. Feedback from the project oversight panel and from the state-of-the practice questionnaire also suggested that these questions merited further study.

Item 3 is concerned with appropriate storage length values. Several projects, some recently completed, would help to support information contained in the Green Book. Item 4, also related to storage length, refers to the capacity of multiple turn lanes, which was a topic better addressed within the double left-turn lanes operational field study.

Objective and Measures of Effectiveness

The objectives for this research study were to determine the following:

- The speed differential for turning vehicles upstream of and at the left-turn taper.
- If speed differential varies based on taper length and posted speed limit.
- Whether the deceleration rates described in the 2011 Green Book are representative of current left-turn drivers.

The proposed measures of effectiveness (MOEs) for the field study were

- Difference in speed of left-turning vehicles between a point upstream of the left-turn lane and the end of the left-turn taper.
- Deceleration rates of left-turning vehicles.

Table 6-1. Green Book material on deceleration lanes where additional research may be needed.

#	2011 Green Book Statements	Potential Additional Research Needs	Potential Source for Information
1	The components of a deceleration lane "consist of the perception-reaction distance, the full deceleration length (also called the maneuver distance), and the storage length (called the queue storage length)."	The distance traveled during perception-reaction time is illustrated on a related figure, but it is not discussed elsewhere. How should this length be determined?	References could be made to one or more previous studies such as (*29*, *80*, *81*) to discuss how to calculate PR distance.
2	"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 speed differential between the turning vehicle and following through vehicles is 10 mph when the turning vehicle 'clears the through traffic lane'. … 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."	Are the assumptions for speed differential rate valid for the current driving population? Are deceleration rates, including using two different rates to calculate the dimension, valid?	Study needed.
3	"The auxiliary lane should be sufficiently long to store the number of vehicles, or queue, likely to accumulate during a critical period. The storage length should be sufficient to avoid turning vehicles stopping in the through lanes waiting for a signal change or for a gap in the opposing traffic flow."	Are storage lengths appropriate?	NCHRP 3-91 developed recommended storage lengths for unsignalized intersections based on data collected as part of that project (*9, 82*) and on equations available in NCHRP Report 457 (*22*). Information on right-turn lane storage is provided in the TRB *Access Management Manual* (*13*) and Stover and Koepke's *Transportation and Land Development* (*83*).
4	"Where turning lanes are designed for two-lane operation, the storage length is reduced to approximately one-half of that needed for single-lane operation. For further information, refer to the HCM."	Is dividing the length needed by 2 for a two-lane turn bay appropriate? What about uneven queuing of vehicles?	Data are available from previous studies, including Sando and Moses (*74*), regarding saturation flow rates for triple left-turn lanes. These studies; however, were not sensitive to factors of interest such as turn lane width, receiving leg width, and downstream friction points. Researching this question was an objective of the Task 4 study on double left-turn lanes.

These variables were used to identify the effects of taper and deceleration length and posted speed limit.

Literature

AASHTO Policies on Deceleration into a Left-Turn Lane

The 2011 Green Book (*2*) provides desirable values for deceleration lengths in Table 9-22 on page 9-126; that table is reproduced in this document as Table 6-2. A copy of the associated figure (Figure 9-48 in the 2011 Green Book) is shown in Figure 6-1. These deceleration length values are the estimated distances needed by drivers to maneuver from the through lane into a turn bay and brake to a stop. They are referenced to a 1998 publication (*84*), but the reference should probably be to the 2003 *Access Management Manual* (*13*, page 172), as that document has similar values and comments about deceleration rates for the portion of the maneuver where the driver is clearing the lane and when the driver is decelerating to a stop. The Green Book discussion also states that "at least part of the deceleration distance for an auxiliary lane assumes that an approaching turning vehicle can decelerate comfortably up to 10 mph before clearing a through lane" and that "a 10-mph differential is commonly considered acceptable on arterial roadways."

The 2004 Green Book (*4*, page 714) and the 2001 Green Book (*85*, page 718) (after corrections from an errata sheet)

Table 6-2. Desirable full deceleration lengths, from Table 9-22 of 2011 Green Book (*2*).

Speed, mph	Distance, ft[a]
20	70
30	160
40	275
50	425
60	605
70	820

Notes:
1. The above full deceleration lengths are $L_2 + L_3$ in Figure 9-48 (see Figure in this document).
2. Assumes a turning vehicle has "cleared the through lane" when it has moved laterally approximately 9 ft so that a following through vehicle can pass without encroaching upon the adjacent traffic lane.
3. The speed differential between the turning vehicle and following through vehicles is 10 mph when the turning vehicle "clears the through traffic lane."
4. 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.

Source: *A Policy on Geometric Design of Highways and Streets* (2011) by the American Association of State Highway and Transportation Officials, Washington, D.C. Used by permission.

had slightly different deceleration lengths; however, the references were also to the 1998 course notes (*84*) referenced in the 2011 Green Book. The deceleration lengths are shown in Table 6-3. The 2004 Green Book and the 2001 Green Book also note that "an approaching turning vehicle can decelerate comfortably up to 10 mph in a through lane before entering the auxiliary lane." The 2001 Green Book was the first policy to provide a value for the amount of speed reduction (10 mph) that can be considered prior to entering the auxiliary lane.

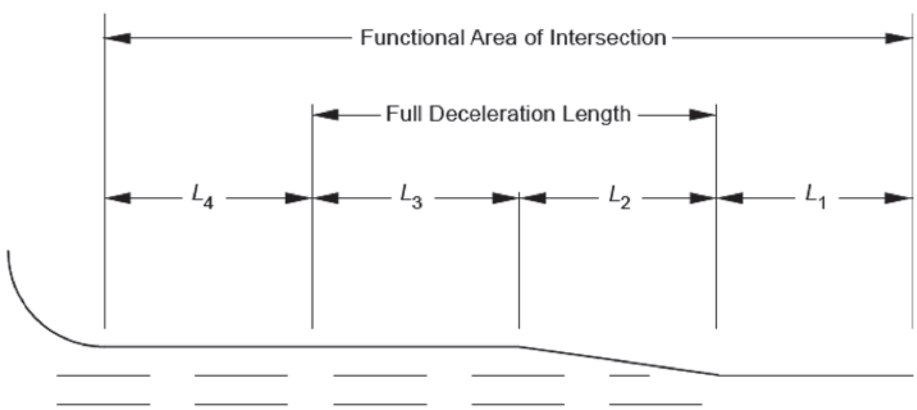
The 1990 Green Book (*86*, page 828) has very different numbers than the more recent versions of the Green Book. Table 6-3 shows the values. The 1990 Green Book states that the total length "required is that needed for a safe and comfortable stop from the design speed of the highway." These lengths "exclude the length of taper which should be approximately 8 to 15 ft longitudinally to 1 ft transversely." The 1990 Green Book also notes that "on many urban facilities it will not be feasible to provide full length for deceleration" and that "in such cases at least a part of the deceleration must be accomplished before entering the auxiliary lane." The 1990 Green Book does not state what part of the deceleration can be accomplished before entering the lane.

The 1984 Green Book (*87*, page 874) and the 1973 *Red Book* (*88*, page 688) base the lengths on average running speed, which is a speed that is less than the design speed of the road. Table 6-3 shows the deceleration lengths for the average running speed. Elsewhere in those documents is information about the assumed relationship between average running speed and design speed, so the assumed design speed for the given average running speed is also shown in Table 6-3. The deceleration lengths include the length of taper. Similar to the 1990 Green Book, these policies note that it will not be feasible to provide the full length for deceleration on many urban facilities, and they do not provide a specific value for the amount of deceleration that can occur prior to entering the auxiliary lane.

Deceleration Rates

Previous sources for deceleration rates are listed below.

- As part of the 1990s study on stopping sight distance (SSD) (*89*), deceleration during braking maneuvers was recorded. The SSD study measured stopping distances for 26 subjects



Notes: L_1 = Distance traveled during perception-reaction time
L_2 = Taper distance to begin deceleration and complete lateral movement
L_3 = Distance traveled to complete deceleration to a stop
L_4 = Storage length

Source: *A Policy on Geometric Design of Highways and Streets* (2011) by the American Association of State Highway and Transportation Officials, Washington, D.C. Used by permission.

Figure 6-1. Illustration of deceleration lane, Figure 9-48 of 2011 Green Book (2).

Table 6-3. Comparison of deceleration lengths from different versions of AASHTO policies.

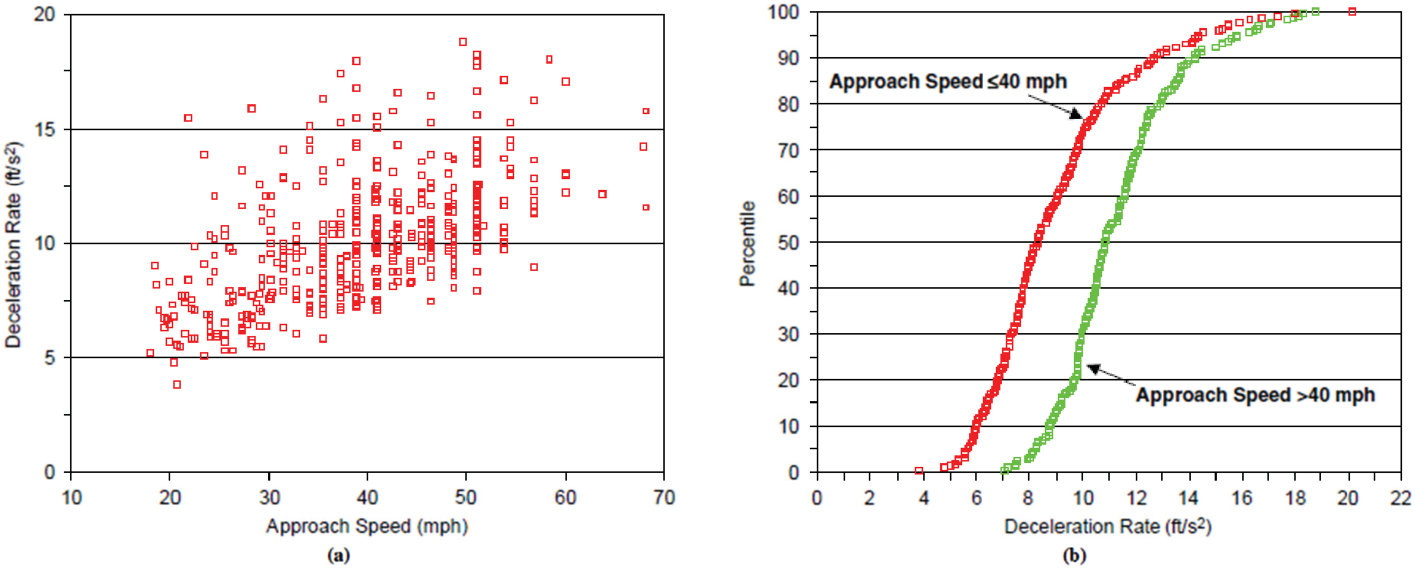
2011 Green Book Deceleration Length Includes Taper Length		2004 Green Book Deceleration Length Includes Taper Length		1990 Green Book Deceleration Lengths Exclude Taper Length			1984 Green Book and 1973 Red Book Deceleration Length Includes Taper Length	
Speed, mph	Decel[a] Dist[b] (ft)	Speed (mph)	Decel Dist (ft)	Speed (mph)	Decel Dist (ft)	Taper Length[c] (ft)	Average Running Speed [Assumed Design Speed] (mph)	Decel Dist (ft)
20	70						20 [20]	160
30	160	30	170	30	235	96-180		
							30 [35]	250
40	275	40	275	40	315	96-180		
		45	340				40 [45]	370
50	425	50	410	50	435	96-180		
		55	485				50 [58]	500
60	605							
70	820							

[a] Decel = deceleration, Dist = distance.
[b] See Table for notes.
[c] Length of taper should be approximately 8 to 15 ft longitudinally to 1 ft transversely; values shown assume 12-ft lanes.

using an initial speed of 55 mph. Test conditions included enabled and disabled antilock brakes, wet or dry pavement conditions, and two geometric conditions (tangent section and horizontal curve) for a total of 986 maneuvers. The deceleration rate used in the 2004 Green Book stopping sight distance procedure (11.2 ft/s^2) was selected based on the results of the 1990s study. A maximum deceleration rate to an unanticipated object was also identified (24.5 ft/s^2) from the SSD study.

- Two studies conducted in the 1980s measured dry pavement deceleration characteristics to traffic signal change intervals. The study by Chang et al. (*90*) found mean decelerations of 10.5 and 12.5 ft/s^2 at the two subject intersections.
- Wortman and Matthias (*91*) found mean deceleration at six study sites of 7.0 to 13.0 ft/s^2. The mean value for all observations from the six intersections was 11.6 ft/s^2, a result consistent with Chang's findings.
- The 5th edition of the ITE *Traffic Engineering Handbook* (*92*) provided a summary of deceleration rates, including deceleration without brakes and representative maximum and comfortable decelerations. The discussion on deceleration without brakes references the same 1940 study used to form the basis of the 1965 Blue Book (*93*) values. The representative maximum braking data provided by ITE in Table 3-12 (*92*) is from a 1948 paper on skid resistance measurements on Virginia highways. It provided data for speeds under 40 mph for four types of dry surfaces and new and worn tires. The guidance in the ITE *Traffic Engineering Handbook* for the higher speeds was back-calculated from the 1994 Green Book stopping sight distance value. The comfortable deceleration advice references a Green Book figure similar to the figure in the 1965 Blue Book (which is based on the 1930s data). A single value of 10 ft/s^2 was also provided as being "reasonably comfortable for occupants of passenger cars."
- The 6th edition of the ITE *Traffic Engineering Handbook* (*94*) provides an update to the guidance on maximum deceleration rates, based on more recent research. It states that the maximum deceleration rate is assumed to be 11.2 ft/s^2, or 0.35 times the acceleration of gravity. Therefore, tire-friction coefficients are no longer provided for calculation of braking distances as they were in the 5th edition of the *Handbook*. The 6th edition contains the same guidance on normal deceleration as the 5th edition, stating that deceleration rates up to 10 ft/s^2 are reasonably comfortable for occupants of passenger cars and that vehicle clearance intervals at traffic signals are determined based on that rate.

A study published in 2007 (*95*) evaluated the deceleration of vehicles between 2.5 and 5.5 sec upstream of signalized intersections at the start of the yellow signal phase, typically considered the "dilemma zone" for drivers. They made video recordings of vehicles approaching signalized intersections and measured several factors for each last-to-go (n=435) and first-to-stop (n=463) vehicle in each lane during each yellow interval, The observed 15th, 50th, and 85th percentile deceleration rates were 7.2, 9.9, and 12.9 ft/s^2, respectively. ITE's recommended comfortable deceleration rate of 10 ft/s^2 (*92*) represented the 52nd percentile for their data, and AASHTO's deceleration rate of 11.2 ft/s^2 for SSD represented the 68th percentile. Gates et al. further analyzed the deceleration rate based on approach speed, using two categories: low (≤40 mph) and high (>40 mph). The correlations between deceleration rate and approach speed are



Source: Gates, T., D. Noyce, L. Laracuente, and E. Nordheim. Analysis of Driver Behavior in Dilemma Zones at Signalized Intersections. In *Transportation Research Record: Journal of the Transportation Research Board, No. 2030*, Figure 4, p. 35. Reproduced with permission of the Transportation Research Board.

Figure 6-2. Effect of approach speed on deceleration rate (95).

presented in Figure 6-2a and distributions for the deceleration rates of the two approach speed categories are plotted in Figure 6-2b. Figure a shows a strong upward trend between deceleration rate and approach speed. Figure 6-2b shows that drivers approaching at high speeds typically used greater deceleration rates than drivers approaching at low speeds. The 15th, 50th, and 85th percentile deceleration rates for stopping drivers approaching at speeds above 40 mph were 9.2, 10.9, and 13.6 ft/s^2, respectively; the rates for stopping drivers approaching at speeds less than 40 mph were 6.4, 8.3, and 11.6 ft/s^2, respectively. The 10-ft/s^2 comfortable deceleration rate recommended by ITE for timing yellow intervals represented the 31st percentile for high speeds and the 74th percentile for low speeds. In other words, they concluded that 69% of stopping drivers approaching at speeds greater than 40 mph will use a deceleration rate greater than the recommended design value of 10 ft/s^2, compared to only 26% of stopping drivers approaching at speeds less than 40 mph. They suggested that, based on their findings, design deceleration values used for determining the length of the yellow interval should be based on approach speed instead of a single default value.

Deceleration Lengths

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. Like the Green Book and many other reference documents, the Texas *Urban Intersection Design Guide* (*12*) states that the length of left-turn lanes depends on three elements:

- Deceleration length.
- Storage length.
- Entering taper.

In many guidelines, deceleration length assumes that some 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. A common design guideline is that, in locations 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.

NCHRP Project 03-91 (*6*) identified recommendations for the approximate total lengths needed for a comfortable deceleration to a stop from the full design speed of the highway. These approximate lengths are shown in Table 6-4 and are based on grades of less than 3%. Two sets of values from NCHRP Project 03-91 are presented in Table 6-4; one set assumes no speed reduction in the main lanes, while the other set accounts for a 10-mph reduction in speed. Researchers acknowledged that, on many urban facilities, it is not practical to provide the full length of deceleration for a left-turn lane, and in other cases, the storage demand consumes part or all of the deceleration length. In either scenario, the length available for deceleration is less than what manuals commonly recommend. Shorter left-turn lane lengths may increase the speed differential between turning vehicles and through traffic, as some deceleration may occur before entering the left-turn lane. Because a 10-mph differential is commonly described as acceptable on arterial roadways, this was included as an alternative in NCHRP Project 03-91, but the

Table 6-4. Deceleration lengths for left-turn lanes (*6*).

Design Speed (mph)	Deceleration Lengths (ft) from Following Sources:				
	Deceleration Lengths from Other Manuals for Comparison			Deceleration Lengths Determined Using 6.0 ft/sec² Deceleration Rate	
	2004 AASHTO Green Book (*4*), page 714	TRB *Access Management Manual* (*13*), page 172	Florida Department of Transportation *2006 FDOT Design Standards* (*14*)	No Speed Reduction in Main Lanes	10-mph Speed Reduction in Main Lanes
30	170	160		170	80
35			145	230	120
40	275	275	155	290	170
45	340		185	370	230
50	410	425	240 (urban) 290 (rural)	460	290
55	485		350	550	370
60		605	405	650	460
65			460	770	550

Note: Blank cells = deceleration length not provided in reference document for the given design speed.

authors recommended that the no-speed-reduction lengths given in Table 6-4 should be accepted as a desirable goal and should be provided where practical.

Warren et al. (*96*) conducted a study on the influence of decelerating right-turning drivers in which they developed a series of equations to predict speeds during deceleration based on the distance from the right turn. In that study, they compared field data with the Green Book's suggestion that crashes become much more common when the speeds in the traffic stream differ by more than 10 mph. By identifying the point at which the difference between the through and the right-turning speeds from the study exceeded 10 mph, they could infer the distance in advance of a driveway at which the full length of the auxiliary right-turn lane should be developed. The taper for entering the deceleration lane would be upstream of this point.

Using speeds from their predictive equations, they calculated the distance in advance of the driveway edge at which the 10-mph difference (based on average speeds of both through and turning vehicles) occurred was 108 ft for the group of study sites with average speeds near 38 mph, 128 ft for sites with speeds near 42 mph, and 151 ft for sites with speeds near 46 mph. Researchers' review of the data showed that typically the standard deviations of turning vehicle speeds did not exceed 4 mph. They then concluded that instead of designing for average speeds, it would be better to design for a 10-mph difference between the average speed of through vehicles and the average minus 4 mph of the turning vehicles. Those resulting lengths were 213 ft for 38 mph, 232 ft for 42 mph, and 237 ft for 46 mph. The researchers noted that these lengths were only for the deceleration lane and did not provide any storage space.

Study Matrix

Table 6-5 shows the initial site selection matrix based on the key site selection variables. The 2011 Green Book specifies two common tapers: a ratio of 8:1 (L:T) for design speeds up to 30 mph and 15:1 for design speeds 50 mph and greater. Although these are defined as straight-line tapers, the lengths (96 ft and 180 ft, respectively, for a 12-ft lane) can be used to compare with other taper designs. Indeed, state design manuals describe a wide variety of designs and lengths, both longer and shorter than the Green Book lengths, many based on speed and others based on turning volumes. For site selection in this study, the equivalent longitudinal length of the taper was measured, and that length was classified based on the values in Table 6-5.

Using this matrix, the ideal outcome would be that the research team would collect two sites for each combination of variables, a total of 16 sites (4 speed limit ranges × 2 taper length categories × 2 sites), as shown in Table 6-5. The research team set a goal of having a minimum of 12 study sites, using Table 6-5 as a guide.

Table 6-5. Potential site selection matrix for deceleration study.

Posted Speed Limit on Major Road (mph)	Taper Length Threshold (ft) for a 12-ft Lane	Number of Sites	
		Below Threshold	At or Above Threshold
30–35	96	2 sites	2 sites
40–45	96	2 sites	2 sites
50–55	180	2 sites	2 sites
60–65	180	2 sites	2 sites

Table 6-6. Controlled site selection characteristics for deceleration study.

Variable	Characteristics
Intersection type	4 legs
Area type	Urban or suburban
Surface condition	Fair to good
Conditions during data collection	Collected only during daytime, daylight conditions, and not raining
Intersection sight distance	Adequate
Traffic control	Signalized; preferred protected-only operations, but sites with permitted turning were allowed
Grade	Preferred between −2 and +2%, but accepted between −4 and +4%
Skew	Approximately 90° in skew angle
Major road number of lanes	4 lanes
Minor road number of lanes	Preferred 4 lanes, but accepted 2 lanes
Left-turn lane assignment	Exclusive left-turn lanes (i.e., no shared lanes)
Approach horizontal alignment	Nominally straight
Departure horizontal alignment	Nominally straight
Adjacent parking lane	None
Nearest upstream friction point	Outside area of influence for left-turn vehicles
Shadowing of left-turn lane	Full
Median type	Raised
Posted speed limit on major road	See Table

In addition to the main study factors (i.e., posted speed limit and taper length), the field study also controlled other important variables that might affect the operation such as those in Table 6-6 so as to limit their effects on the MOEs and to allow the effects of posted speed limit and taper length to be more easily identified. The variables expected to affect the operation, but not significantly change the effectiveness of deceleration lanes, do not necessarily need to be included in the analysis as study factors, but it is still useful to control them by holding them at a constant level. Table 6-6 lists the variables controlled during site selection. Additional geometric and traffic control device variables were collected in the field; researchers did not anticipate including all of these variables in the models for analysis, but they were available if additional investigation was needed to help explain the results for a specific site. Table 6-7 lists additional variables collected at each study site.

Study Sites

The research team used the characteristics in Table 6-5 through 6-7 to select study sites. By using these characteristics to guide the site selection process, a more efficient and economical data collection plan was developed while ensuring that important effects of key characteristics could be estimated. A tradeoff to using this approach was that considerably more resources were required to identify potential study sites, as opposed to choosing study sites arbitrarily.

Some potential study sites were suggested by panel members or through the responses to the state-of-the-practice questionnaire. The research team considered Texas, Ohio, Illinois, California, North Carolina, Florida, Tennessee, Louisiana, Mississippi, and Alabama as possible focus states for study sites. The team contacted colleagues and practitioners in the selected states to obtain further information and identify additional potential deceleration sites. Reviewing these sites in aerial photographs available online, researchers gathered preliminary information on site characteristics to select the sites for the study, taking into consideration the states that would enable the most efficient means of completing data collection efforts.

The goal was to collect data in a minimum of three states. Based on the information available, researchers decided to focus their efforts on Texas and a selection of states in the southeastern part of the country. A more detailed review of

Table 6-7. Additional variables in deceleration study.

Variable	Comments
State	A minimum of three states represented
Left-turn lane width	Preferred 12 ft, but accepted as low as 9 ft for low-speed sites
Through lane width	Lane width measured and noted
Deceleration length	Desired an equal number of sites with lengths greater than and less than those recommended by the Green Book, but most sites had shorter lengths
Pedestrian activity	Level of activity observed and noted
Bicycle activity	Level of activity observed and noted
Heavy vehicles	Only considered passenger cars

Table 6-8. Final site selection matrix for deceleration study.

Posted Speed Limit on Major Road (mph)	Taper Length Threshold (ft) for a 12-ft Lane	Number of Sites	
		Below Threshold	At or Above Threshold
30–35	96	2 sites	2 sites
40–45	96	2 sites	2 sites
50–55	180	1 site	3 sites
60–65	180	2 sites	1 site

potential sites in Alabama, Florida, Mississippi, North Carolina, Tennessee, and Texas was conducted to produce a list of viable candidate sites. More than 70 sites in these six states were considered during site selection, and a list of candidate sites was developed for researchers to visit in person and collect data at the most promising sites. Researchers ultimately visited 42 of these sites in Texas, Mississippi, Alabama, and Florida, choosing 15 of them for data collection. Table 6-8 shows the final distribution of those 15 sites.

Two of the sites (one 50 mph, below threshold; one 65 mph, below threshold) had to be eliminated after site selection and initial data collection because the video files recorded at those sites did not produce images of the clarity needed to accurately obtain the necessary data. Another site (55 mph, above threshold) was eliminated because no counter data were recorded during data collection. Ultimately, data reduction and analysis were completed for 12 sites. Table 6-9 lists the key study site characteristics for the deceleration lane study sites. Key length dimensions in Table 6-9 and key locations in the data collection protocol are shown in Figure 6-3. Those key locations were along the length of the left-turn lane to establish fixed points in the site characteristics documentation. The key locations used in post-processing and reducing the data from the video included

A. Upstream of the turning lane (location of the automated traffic counter to collect free-flow or pre-deceleration, speeds).
B. Beginning of the taper.
C. End of the taper.
D. End of the deceleration length, as recommended by the Green Book.
E. End of the turning lane at the stop line.
F. Other locations as needed.

Data Collection

The primary data collection method used was video recording, supplemented by GPS receivers, lidar emitters (i.e., laser speed guns), and automated speed counters. Video pro-

Table 6-9. Site characteristics at study sites.

Site #	City	PSL	Len_Taper	Taper_Thresh	Len_Decel	Len_LTL	LW_Thru	LW_LT	Med_Wid	Sig_Op
AL-03	Mobile	50	186	Above	180	180	11.0	11.0	18.5	Pro/Per
AL-08	Mobile	35	115	Above	230	245	12.0	11.3	18.0	Pro/Per
AL-09	Mobile	35	61	Below	94	94	8.5	9.0	14.0	Per Only
FL-03	Tallahassee	45	98	Above	365	380	11.5	10.5	4.0	Pro/Per
FL-09	Tallahassee	35	80	Below	173	173	10.5	10.0	5.0	Pro/Per
FL-10	Tallahassee	45	71	Below	216	216	11.5	9.0	5.0	Pro Only
MS-03	Biloxi	45	93	Below	186	186	11.5	10.5	11.3	Pro/Per
MS-05	Biloxi	35	125	Above	80	80	12.0	11.0	13.0	Per Only
MS-08	Gulfport	45	119	Above	255	255	11.0	10.0	5.5	Pro/Per
TX-21	Pflugerville	50	204	Above	115	115	12.5	11.5	29.0	Pro Only
TX-28	Austin	65	191	Above	283	283	13.5	11.5	56.0	Pro Only
TX-33	Austin	65	152	Below	312	312	13.0	11.5	56.0	Pro Only

Note: PSL = posted speed limit (mph); Len_Taper = length of taper (ft); Taper_Thresh = whether Len_Taper is above or below the taper length threshold in Table ; Len_Decel = length of deceleration lane (ft); Len_LTL = length of left-turn lane from end of taper to stop bar (ft), includes deceleration length + storage length; LW_Thru = width of through lane (ft); LW_LT = width of left-turn lane (ft); Med_Wid = width of median (ft); Sig_Op = left-turn signal operation mode: (Pro)tected, (Per)mitted, or both (Pro/Per).

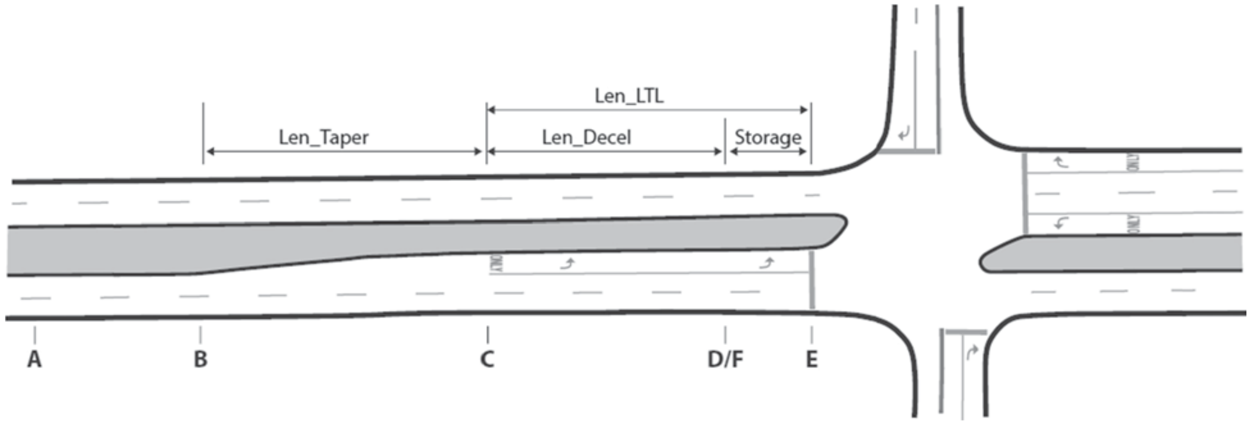


Figure 6-3. Key dimensions of left-turn lanes at study sites.

vided a permanent recording of the conditions and events at the site, speed counters provided the vehicle speeds upstream of the site, and the GPS and lidar provided a way to obtain comparative speed data to use for quality control. Traditionally, video is not as commonly used to collect this type of speed data as the other methods, but the research team conducted preliminary field tests and compared the relative benefits of each method in selecting the video method for this study. Table 6-10 provides a summary comparison of the data collection methods considered—video had several key advantages: it could provide a permanent visual record of events during data collection, it produced both speed and volume measurements, and it did not require researchers or equipment to be within the roadway. These advantages outweighed the limitations of video. Ultimately, a combination of the four methods was used to collect the data, but video was the primary data source.

Prior to beginning data collection, data collectors visited the site to determine the best locations to set up equipment, and they coordinated as needed with local authorities and

Table 6-10. Comparison of data collection methods in deceleration lane field study.

Data Collection Method	Advantages	Limitations
Video	• Provides a permanent record of conditions at the site • Provides a method of obtaining accurate volumes • Permits observation of all vehicles at the site, and a larger number of vehicles in general • Can be inconspicuous and not affect traffic	• Requires a vantage point with sufficient field of view to capture the entire turning lane • Precision is related to the frame rate, resolution, and perspective of the camera and recorder • Trailer must be towed from site to site
Automated traffic counter (piezoelectric or tube sensor)	• Records speeds and classifications of all vehicles • Relatively easy to transport and install on collector street (but perhaps not high-speed arterials)	• Records speeds at only one point • Installed in the roadway; drivers must travel over the sensors
Lidar	• Records a speed-distance profile • Can be used for all vehicles • Easy to transport	• Proper positioning may be difficult; line of sight can be obstructed by other traffic, which is common when studying left-turn lanes • Operator needs to be inconspicuous to avoid influencing driver behavior
Instrumented vehicle	• Records a detailed speed-location profile using GPS • Can obtain data on drivers at multiple locations	• Only a limited number of subject drivers can be accommodated within the time and budget available; related issues include: ○ Range of driver characteristics (age, experience, etc.) ○ Drivers must be scheduled, as well as compensated for their time

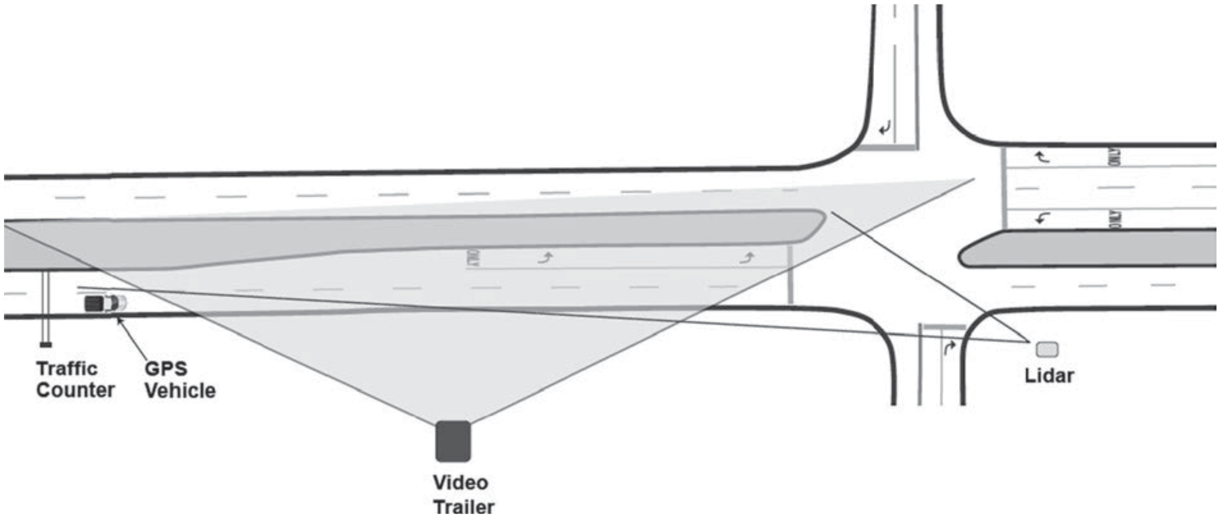


Figure 6-4. Schematic drawing of equipment setup at study site.

adjacent landowners. The length of the study period at each site was generally between 3 and 4 hr. Figure 6-4 illustrates the position of the video trailer and other equipment at an example study site.

Video Trailer

Researchers used a trailer with a telescoping mast that raised two cameras up to 30 ft above ground level, positioned with a field of view perpendicular to the left-turn lane. These cameras were connected to a digital video recorder, which created a time-stamped video recording of traffic conditions at each intersection during the study period. During the setup process at each study site, researchers identified the key locations shown in Figure 6-3.

During the setup process, researchers temporarily placed traffic cones or other markers at key locations (B, C, D, E, and F in Figure 6-3) to provide visual cues on the video recording. At most study sites, the left-turn lane did not provide the full deceleration length as recommended by the Green Book, so point D was not available. As a result, a point F was created, typically 50 or 100 ft upstream of the stop line, to provide an intermediate point between points C and E.

The video trailer was typically positioned on the roadside near the end of the taper, which was often the center of the field of view needed to capture the entire length of the left-turn lane and some additional distance upstream and downstream. The two cameras were aimed to capture side-by-side points of view, in effect creating a "landscape" image of the left-turn lane, as shown in Figure 6-5. In some cases,

Figure 6-5. Screenshot of left-turn lane video.

the length of the left-turn lane or the available roadside area to position the trailer mandated that the cameras be aimed to exclude a middle portion of the left-turn lane, typically between points C and F. Using this method, researchers were able to maintain sufficient visibility on the extents of the left-turn lane and key points while omitting a short length from the field of view that was not necessary for data reduction. Figure 6-4 shows the trailer positioned on the same side of the road as the left-turn lane, though site conditions often made it more suitable to position the trailer on the opposite side of the road to obtain the best vantage point.

Automated Traffic Counter

Researchers also installed an automated traffic counter upstream of the left-turn lane, to collect free-flow speed data on vehicles approaching the intersection. The counter system used pneumatic tubes on the road surface to make contact with the tires of approaching vehicles; the tubes were connected to the counter itself, which was placed on the roadside or in the median out of approaching drivers' field of view. The location of the counter was chosen so that vehicles would be far enough upstream of the intersection to not be affected by actual or anticipated deceleration to enter the left-turn lane. Thus, the speed data collected by the counter would closely approximate not only the free-flow speed of vehicles on the major road, but also the expected speed of through vehicles as they approached the beginning of the left-turn taper.

Quality Control Speed Data

After the video recording equipment was set up and activated, researchers collected additional speed data for use as quality control comparisons for the speed data that would be obtained from the video. First, the researchers used a GPS receiver connected to a laptop computer to record speed profiles in the research vehicle, a white four-door Ford F-150 with the TTI logo on both front doors. The researchers completed 20 trips through the study site in the research vehicle, 10 trips at the posted speed limit and 10 more trips at 10 mph below the posted speed limit. Half of the trips were terminated with a left turn, and half were conducted by continuing through the intersection in the left-hand through lane. This distribution provided various conditions under which speed data recorded with the GPS receiver could be compared to the speed data reduced from the video. Researchers established and maintained speed by using the vehicle's cruise control on the approach to the intersection.

After the GPS data were collected, researchers collected speed data using a lidar gun connected to a laptop computer. The lidar gun enabled collection of speed-distance profiles on vehicles other than the research vehicle. Researchers identified a location away from the left-turn lane to set up the computer and lidar gun to record a sample (typically between 10 and 20) of left-turning and through vehicles. This location was chosen to minimize any effects that the researcher's presence might have on approaching drivers. As shown in Figure 6-4, the lidar position could be downstream of the intersection if sufficient lines of sight and setup space were available. The lidar position could also be established upstream of the left-turn lane if site characteristics were more conducive there. The lidar gun collected speed and distance data on vehicles approaching the intersection at a frequency of approximately 0.3 sec; the data were stored on the laptop computer in a text-based log file, which was later converted to a spreadsheet for use in data reduction.

Site Characteristics

Finally, the researchers documented conditions at each study site by completing a site characteristics worksheet, recording key measurements such as lane and median widths and distances from the stop line to points A through F, and taking photographs of key locations and features along the approach to and departure from the left-turn lane.

Data Reduction

Researchers reduced the data from the video, lidar, GPS receiver, and traffic counter to determine speeds of turning vehicles at the key locations at each of the study sites. Vehicle speed within the left-turn lane was determined by calculating the time it takes for a vehicle to travel a known distance. Key locations were identified within the turning lane as described previously in the Data Collection section, the distances between those points were measured and recorded, and a reviewer watching the video noted the times when subject vehicles reached each of those locations, to calculate the elapsed time to travel those known distances.

The critical characteristic of this study is that the known distances are very short, so researchers must be able to determine the elapsed time with great precision; otherwise, calculated speeds may be highly inaccurate. To use video effectively on this project, researchers recorded images of the entire left-turn lane within a single field of view, and a video recording system was used with sufficient resolution for the video reviewer to determine when a specific point on a subject vehicle (e.g., the front tire or the front bumper) reached a key location within the lane, based on the timestamp recorded with the video image. The recording system also could play back frame-by-frame to pinpoint when vehicles arrived at key locations.

The research team developed a spreadsheet in which to document the details of left-turn deceleration events. The reviewer then opened the video file and, using field notes recorded at the study site, identified the location of each key point on the video screen. Field technicians at each site walked across the roadway at each key point, which was recorded in the video. Technicians then noted when they walked across the roadway. Based on that information, the video reviewer watched the video at that time, and, using a transparency film overlaid on the video screen, drew lines corresponding to each key point, similar to the yellow lines superimposed on the image in Figure 6-6.

The reviewer then watched the video to identify appropriate target vehicles. Target vehicles were passenger vehicles that were the first in queue and came to a complete stop before making the left turn. Vehicles that did not come to a complete stop before turning were not accepted as target vehicles. After a suitable left-turning vehicle was identified in the video, the video was reversed until the target vehicle was upstream of the first line at point B. Using the timestamp on the screen and the frame-advance feature in the playback software, the reviewer then identified the hour, minute, second, and frame in which the target vehicle reached point B. That information for the first target vehicle at a given site was recorded in the spreadsheet. That process was repeated for points C, F, and E. The equations in the spreadsheet calculated the average speed of the vehicle between successive points, based on the elapsed time to travel those distances. The video reviewer continued to identify and document the information on left-turning vehicles until 100 left-turning events were documented or until 4 hr of video were reviewed.

To add another level of quality control, the video files were reviewed a second time, by a second reviewer, using a second playback program that provided frame count instead of a clock counter. This provided an additional confirmation that the results obtained in the video review were accurate. The speed data from the second review was added to the spreadsheet, and equations in the spreadsheet calculated the difference in speed values with the first video review.

Researchers then reduced the files from the GPS receiver to include in the spreadsheet. While completing the GPS trips at each site, researchers added another trip to stop and mark the location of each key point in the GPS speed-distance profile. While reducing the GPS data, researchers used the latitude/longitude coordinates of the four key points in the speed-distance profile to identify those points in the other 20 trips. The cumulative distance for the GPS reading closest to each key point was identified, and the elapsed time and average speed over those distances were calculated. The average speed values were entered into the spreadsheet, and differences between GPS speeds and video speeds were calculated.

Finally, researchers reduced the speed-distance data from the lidar gun for inclusion in the spreadsheet. The data from the lidar gun was recorded in a format similar to that of the GPS data, in that it contained speed and location of each reading. However, the lidar gun records at a frequency of approximately 0.3 sec, so there was typically not a speed reading at the exact location of all four key points for a given target vehicle. Researchers interpolated the elapsed time between the key points using speed-distance readings from adjacent points. That interpolated elapsed time was used to calculate the average speed over the known distance between key points. Those speed values were entered into the spreadsheet, and differences between GPS speeds and video speeds were calculated.

Researchers then processed the data from the traffic counter files to add to the database. Each vehicle recorded by the traffic counters was recorded with a timestamp in addition to its speed and wheelbase characteristics. The timestamp and wheelbase information was compared to the video to identify and match vehicles in the counter and video data files. Using the time and speed at which a particular type of vehicle crossed the counter, researchers could estimate when that

Figure 6-6. Example of video showing locations of key points.

vehicle would be visible in the video. To identify a particular left-turning vehicle documented in the video, researchers established the presence of one or more nearby vehicles; focusing on identifying non-passenger vehicles (e.g., buses and semi-trailers), researchers could confidently establish the elapsed time between the counter reading and the appearance in the video. Researchers could then determine the number of vehicles in the video between that non-passenger vehicle and the left-turning vehicle previously documented. Knowing the likelihood that the same number of vehicles traveled through the counter in the same order upstream of the left-turn lane, researchers compared timestamps in the counter file for vehicles near the non-passenger vehicle and determined which counter vehicle corresponded to the left-turning vehicle in the video. The speed associated with that vehicle was then entered into the database as the upstream free-flow speed for the left-turning vehicle.

For each vehicle, researchers indicated their confidence level that they had identified the same vehicle in both the counter and video. For about one-fifth of the vehicles in the video (200 of 684), researchers were not able to identify the corresponding vehicle in the counter file with high confidence, and upstream free-flow speeds for those vehicles were not included in the database. In 195 of those 200 cases, there was no counter record of the vehicle because the counter was inactive when the vehicle traveled through the left-turn lane. In the other five cases, no appropriately sized vehicles traveled through the counter within the probable timeframe that would correspond to the left-turn vehicle that appeared in the video; those vehicles likely entered the roadway from a driveway or cross street between the counter and the left-turn lane, so there was no counter record of those vehicles. Upstream speeds for the other 484 vehicles were identified and included in the spreadsheet for analysis.

Quality Control

That each of the speed data sources (i.e., two video reviews, GPS, and lidar) calculated speeds in a different way enabled thorough quality control checking, but it increased the likelihood that speeds from different sources would not be equal, so in the comparison of data, researchers set a threshold of 3.0 mph as the target for maximum difference in speeds. Given that each method of collecting data has its own margin of error, which could be cumulative when comparing data from multiple sources, a value of 3.0 mph seemed a reasonable goal. No left-turn event had data from every source, but each quality control (QC) comparison had at least two sources of data.

When each site's data was compared side by side in a spreadsheet, researchers reviewed the data looking for differences larger than 3.0 mph. When they were identified, researchers reviewed the video data and the second (or third) source to identify any adjustments needed. In many cases, the difference in speed value was due to data entry (e.g., typing in the wrong second or frame for video data) or spreadsheet miscalculation (e.g., an equation in Excel referred to the wrong value). In some cases, the output from the GPS or lidar produced an average speed higher or lower than all of the individual speed readings collected over that distance, indicating a calculation adjustment was needed. If the source of the discrepancy could be identified, it was corrected, but in some cases, the source was unknown. In those instances, researchers either used the average of the individual speed readings over that distance or they removed that data point from the comparison.

Researchers also checked the speeds from the upstream counter in relation to the speeds in the taper and found that 74 of the 484 vehicles had a negative speed difference; that is, the vehicles increased speed between the counter and the taper. In reviewing the characteristics of those vehicles, researchers determined that those vehicles had likely entered the study site from a driveway or side street near the counter and had not reached free-flow operating speed when passing through the counter. Those 74 vehicles were removed from the analysis, leaving 410 vehicles at 12 sites to be considered.

After all of the quality control process was completed, the speed data for all of the target left-turning vehicles obtained from the video were compiled into a single database for use in analysis.

Analysis

The 2011 Green Book provides desirable values for deceleration lengths in Table 9-22 (page 9-126); that table is adapted for this document as Table 6-2. The research team focused its analysis efforts on three key guidelines that the Green Book provides as notes in conjunction with Table 9-22:

1. The speed differential between turning vehicles and following through vehicles is 10 mph when the turning vehicle clears the through-traffic lane (see Note 3 in Table 6-10).
2. The values for deceleration length are based on a 5.8 ft/s^2 average deceleration while moving from the through lane into the left-turn lane (see Note 4 in Table 6-10).
3. The values for deceleration length are based on a 6.5 ft/s^2 average deceleration after completing the lateral shift into the left-turn lane (see Note 4 in Table 6-10).

The purpose of the analysis was to determine whether those three guidelines represented the actual deceleration of left-turning vehicles observed at the study sites. Researchers used multiple approaches to analyze the deceleration data, performing basic analytical reviews on the data in a spreadsheet and in graphs to look for possible patterns and trends and then conducting a more extensive statistical analysis using the ANACOVA mixed model. The variables reviewed in these analyses are listed in Table 6-11.

Table 6-11. Variables included in deceleration data analysis.

Variable	Variable Type	Description
Spd_DiffAC	Response	A observed left-turning vehicle's difference in speed (mph) between the upstream counter at point A and the end of the taper at point C in Figure 6-3.
Decel_AC	Response	The deceleration rate of an observed left-turning vehicle (ft/s^2) between the upstream counter at point A and the end of the taper at point C.
Decel_CE	Response	The deceleration rate of an observed left-turning vehicle (ft/s^2) between the end of the taper at point C and the stop line at point E.
PSL_Major	Site (Key)	The posted speed limit (mph) on the major road at the approach to the left-turn lane.
Len_Taper	Site (Key)	The length of the left-turn taper (ft); the distance between points B and C in Figure 6-3.
Len_Decel	Site (Key)	The length of the deceleration lane (ft); the distance between points C and E in Figure 6-3.
Len_T+D	Site (Key)	The sum of the taper and deceleration lengths (ft).
LW_Thru	Site (Secondary)	The width of the through lane adjacent to the left-turn lane (ft).
LW_LT	Site (Secondary)	The width of the left-turn lane (ft).
Site	Random	The study site at which the data were collected.
Spd_BC	Individual Vehicle	The average speed of the left-turning vehicle in the left-turn taper (mph).
Spd_A	Individual Vehicle	The speed of the left-turning vehicle at the counter upstream of the left-turn lane (mph).

Speed Differential of Left-Turning Vehicles

The first research question in this study focused on left-turning vehicles' difference in speed between the upstream traffic counter and the end of the taper. To determine whether the differential from Note 3 in Table 6-10 is representative of current drivers, researchers analyzed the differences in speed between the upstream traffic counter and the left-turn taper for the 410 left-turning vehicles observed in the video that had corresponding speed readings from the traffic counter. The speed at the upstream traffic counter provides an indication of the speeds of through vehicles in addition to enabling same-vehicle comparisons of speed and deceleration for the observed left-turning vehicles.

Figure 6-7 shows the cumulative distribution of the speed differential data (Spd_DiffAC). About 46% of all observed vehicles had a speed differential less than the Green Book suggested value of 10 mph, but the distributions varied greatly by posted speed limit. Approximately two-thirds of the vehicles observed at posted speed limits of 35 and 45 mph slowed by 10 mph or less, compared to just 7% of vehicles on 50-mph roadways and 34% of those on 65-mph roadways. About 30%

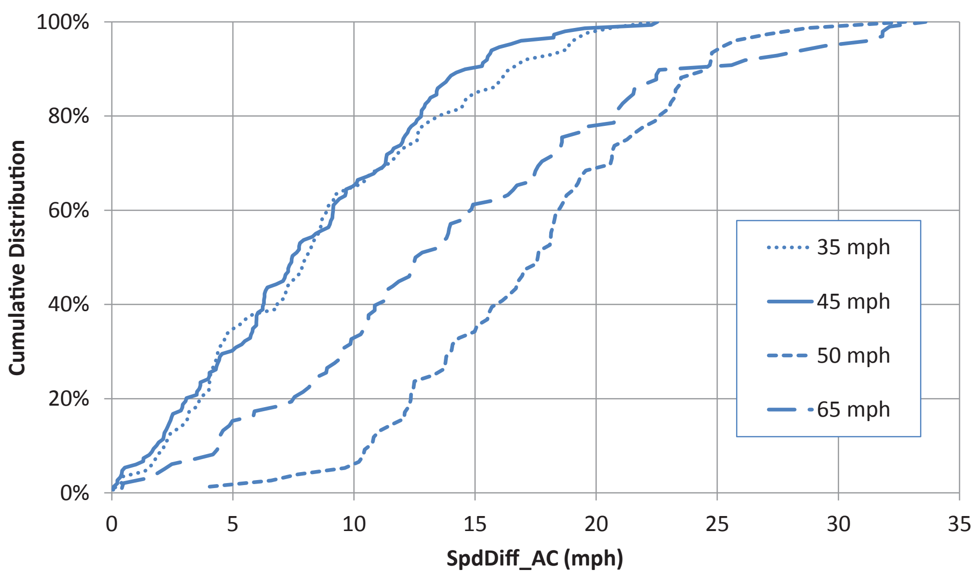


Figure 6-7. Cumulative distribution of speed difference between upstream and taper.

Table 6-12. Speed differential by site.

Site	Taper Length (ft)	Decel Length (ft)	Posted Speed Limit (mph)	Speed Differential (mph)		Number of Vehicles
				Average	Std Dev	
AL-08	115	230	35	7.4	3.4	17
AL-09	61	94	35	12.9	5.1	17
FL-09	80	173	35	4.7	2.7	33
MS-05	125	80	35	12.9	5.2	20
FL-03	98	365	45	10.2	5.4	38
FL-10	71	216	45	4.8	3.8	28
MS-03	93	186	45	9.4	4.5	24
MS-08	119	255	45	7.9	4.7	59
AL-03	186	180	50	15.4	4.4	44
TX-21	204	115	50	20.3	5.5	32
TX-28	191	283	65	18.1	6.8	55
TX-33	152	312	65	8.9	5.9	43
All Sites				11.4	7.0	410

of the left-turning vehicles at the 50 mph speed limit and 21% at the 65 mph speed limit slowed by more than 20 mph. Table 6-12 lists the speed differentials by study site, ordered by speed limit.

While posted speed limit is often used in comparisons to other variables, the difference in speed between the upstream counter and the taper could be influenced by some factors. To properly account for the combinations of these variables, researchers used the ANACOVA mixed model to analyze the data. The model that produced the most informative results is summarized in Table 6-13.

Table 6-13 reveals that the speed differential was significantly and positively related to the upstream speed (Spd_A). This is intuitive, because one can expect the speed differential to rise as the upstream speed rises, and a vehicle's speed differential cannot be greater than its initial speed. A plot of the two variables is shown in Figure 6-8, and a summary of the data in 10-mph increments is shown in Table 6-14. All of the speed differentials greater than 30 mph and 45 of the 50 differentials greater than 20 mph occurred for vehicles with upstream speeds of at least 50 mph. Speed reductions between 10 and 20 mph occurred commonly throughout all initial speed ranges except those less than 30 mph or greater than 69 mph.

Results Compared to Green Book Guidance

Comparing the results in Figure 6-8 and Table 6-14 to the Green Book guidelines provides opportunity for some interesting insights. Supporting the desirable deceleration lengths

Table 6-13. Evaluation of difference between upstream speed and speed at the end of the taper.

Response Spd_DiffAC

Summary of Fit

RSquare	0.663367
RSquare Adj	0.658355
Root Mean Square Error	4.137866
Mean of Response	11.41751
Observations (or Sum Wgts)	410

Parameter Estimates

Term	Estimate	Std Error	DFDen	t Ratio	Prob>|t|
Intercept	-7.688556	23.07555	6.06	-0.33	0.7502
PSL_Major	-0.096517	0.233616	6.129	-0.41	0.6936
Len_Taper	0.017966	0.061903	6.071	0.29	0.7813
Len_Decel	-0.032595	0.020219	6.097	-1.61	0.1573
LW_Thru	-2.167737	1.849077	6.159	-1.17	0.2844
LW_LT	2.4297784	3.197241	6.078	0.76	0.4757
Spd_A	0.5955358	0.041732	398.4	14.27	<.0001*

REML Variance Component Estimates

Random Effect	Var Ratio	Var Component	Std Error	95% Lower	95% Upper	Pct of Total
Site	0.8811868	15.087623	9.0000753	-2.552525	32.727771	46.842
Residual		17.121935	1.2151922	14.969187	19.777535	53.158
Total		32.209558				100.000

-2 LogLikelihood = 2370.7763692

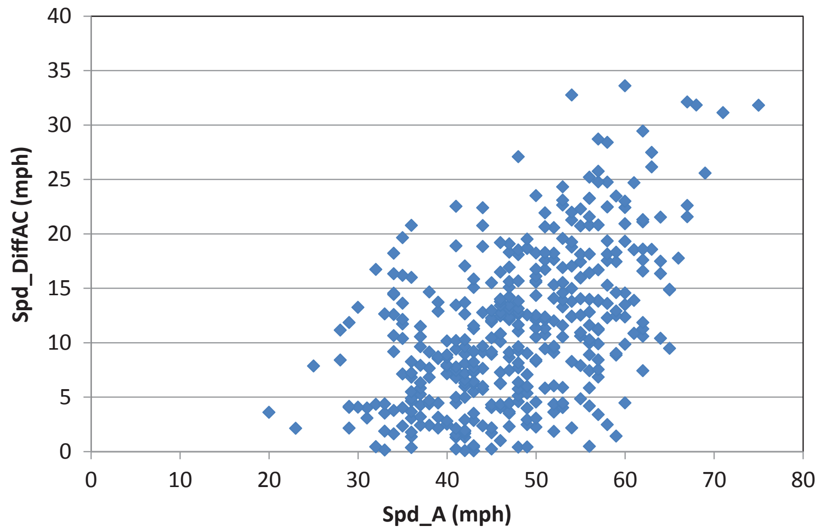


Figure 6-8. Plot of speed differential versus upstream speed.

and notes in Table 9-22, the Green Book (*2*) adds the following text on page 9-127:

> 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 before clearing a through lane. Shorter auxiliary lane lengths will 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.

Two key items are discussed in this selection of text: the roadway classification and the length of the auxiliary lane. All of the sites in this study were arterials, so the speed differential on collectors cannot be tested in this study; however, the Green Book's observation that higher speed differentials on collectors might be acceptable due to slow speeds does not appear to be consistent with what was observed on the lower-speed arterials in this study. In fact, the higher speed differentials in this study occurred for vehicles traveling at higher speeds. A key descriptor in the Green Book text is the word "acceptable," which is not the same as "expected." Based on the data from this study, one would not expect higher speed differentials on streets with lower speeds, even if it might be considered acceptable for purposes of design, operations, or safety.

The first edition of the *Access Management Manual* (*13*) and the draft second edition (*97*) both cite research by Soloman (*98*) stating that crash potential increases as the difference in speed increases between vehicles in a traffic stream. In particular, the research concludes that a vehicle traveling 10 mph slower than other traffic (i.e., a vehicle with a 10-mph speed differential) is twice as likely to be involved in, or cause, a crash as when all vehicles are traveling at the same speed, and the likelihood of a crash increases exponentially with greater speed differentials. This research is used in the *Access Management Manual* as the basis to recommend deceleration lengths that include a 10-mph speed differential. Similarly, Soloman's research and/or the *Access Management Manual* can also be referenced in the Green Book to support the 10-mph differential as "acceptable"; the existing text that describes the assumption of comfortable deceleration for the driver could be replaced with a description of the benefits of reducing the likelihood of a crash.

Table 6-14. Speed differential by upstream speed.

Upstream Speed (mph)	Number of Vehicles with a Speed Differential of…				
	0–10 mph	10–20 mph	20–30 mph	> 30 mph	Total
20–29	7	2	0	0	9
30–39	47	21	1	0	69
40–49	93	54	4	0	151
50–59	38	72	26	1	137
60–69	4	22	13	3	42
≥ 70	0	0	0	2	2
Total	189	171	44	6	410

Table 6-15. Evaluation of speed differential with combined taper and deceleration length.

Response Spd_DiffAC

Summary of Fit

RSquare	0.663342
RSquare Adj	0.659175
Root Mean Square Error	4.137816
Mean of Response	11.41751
Observations (or Sum Wgts)	410

Parameter Estimates

Term	Estimate	Std Error	DFDen	t Ratio	Prob>ltl
Intercept	-24.42452	14.8144	7.15	-1.65	0.1423
PSL_Major	0.0313606	0.189744	7.391	0.17	0.8732
Len_T+D	-0.037865	0.01932	7.111	-1.96	0.0902
LW_Thru	-2.601044	1.780614	7.172	-1.46	0.1865
LW_LT	4.6833316	2.130006	7.079	2.20	0.0634
Spd_A	0.5981215	0.041689	401.1	14.35	<.0001*

REML Variance Component Estimates

Random Effect	Var Ratio	Var Component	Std Error	95% Lower	95% Upper	Pct of Total
Site	0.8697232	14.890988	8.2216983	-1.223541	31.005517	46.516
Residual		17.121525	1.215134	14.968874	19.776991	53.484
Total		32.012513				100.000

-2 LogLikelihood = 2367.6496048

Referring to the length of the auxiliary lane, a relationship between shorter lengths and higher speed differentials, as stated in the previous Green Book quotation, is intuitive. Indeed, the two largest speed differentials are associated with sites that have the greatest differences in length compared to Green Book recommendations; however, a strong statistical relationship did not appear in this dataset, as shown in Table 6-15 and Figure 6-9. There were various speed differentials for each deceleration lane in the study. Neither taper length nor deceleration length was significant at $\alpha = 0.05$ in any of the statistical models explored, and the two length variables had coefficients with small values and opposite signs. Other models looked at taper and deceleration length together, and the result was that an increase of 100 ft in combined length would decrease the speed differential between 3.0 and 4.0 mph, though that relationship was not statistically

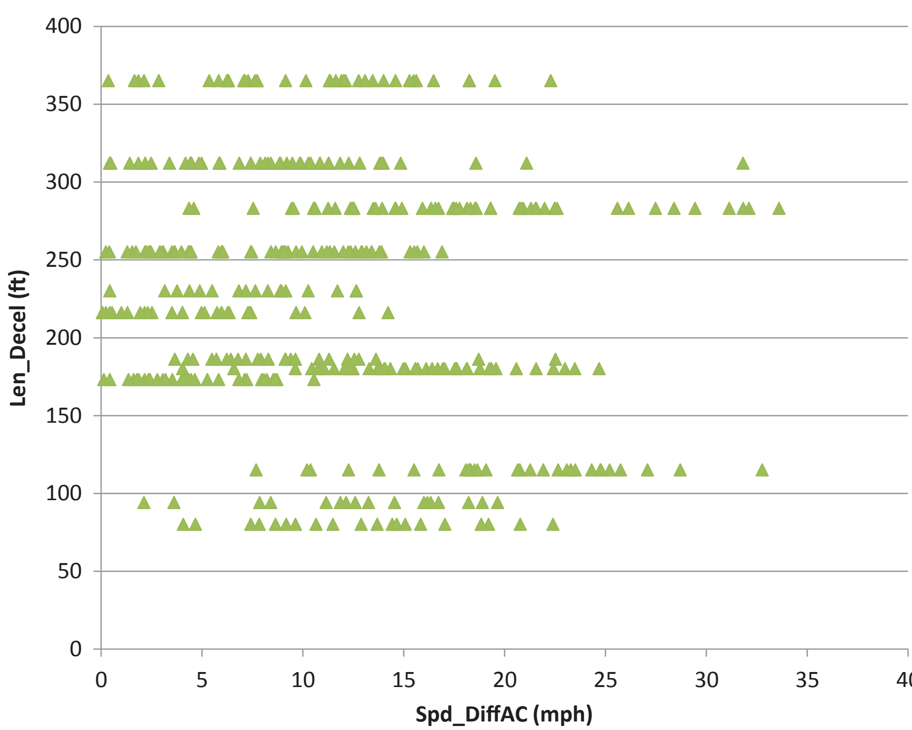


Figure 6-9. Plot of deceleration length versus speed differential.

significant at $\alpha = 0.05$ (see Table 6-15). Note that although Len_Decel was not significant at $\alpha = 0.05$, the sign of its coefficient makes sense and the effect may become significant as the number of sites increases, so the general effect of deceleration length appears to be consistent with the Green Book text, but its practical effect is not as apparent.

Deceleration of Left-Turning Vehicles

The other key research question in this study focused on the deceleration rate of left-turning vehicles. In the notes associated with desirable deceleration lengths, the Green Book describes deceleration rates of 5.8 ft/s^2 while moving from the through lane into the turn lane and 6.5 ft/s^2 after completing the lateral shift into the turn lane. Researchers compared the upstream speeds and end-of-taper speeds of the observed left-turning vehicles and, using the time it took them to travel to the end of the taper, calculated deceleration rates between points A and C (see Figure 6-4). Similarly, speeds at the end of the taper were compared with the time spent decelerating to a stop at the stop line to calculate deceleration rates within the deceleration lane. The analysis of those deceleration rates is described in this section.

Deceleration Upstream of the End of the Taper

The first deceleration to be considered is that which occurred as vehicles prepared to enter the left-turn lane; this deceleration took place upstream of and within the taper. Table 6-16 lists the deceleration rates prior to the end of the taper at each study site, with the sites ordered by taper length. The Green Book deceleration rate of 5.8 ft/s^2 is within the range of average rates at the study sites, but 8 of the 12 sites had an average rate higher than 5.8 ft/s^2. In addition, the 15th percentile deceleration rate for all sites was 2.8 ft/s^2, and two sites' 15th percentile rate was equal to or greater than 5.8 ft/s^2.

For reference, each site's taper rates are also shown in Table 6-16; the distribution of taper rates is similar to the Green Book's suggested range of rates (i.e., 8:1 at lower speeds and 15:1 at higher speeds). Five of the eight sites below 50 mph had taper rates less than 10:1, and all four high-speed sites had rates greater than 13:1.

Figure 6-10 shows the cumulative distribution of the deceleration rates in and upstream of the taper (Decel_AC). Overall, about 40.5% of the observed left-turning vehicles had a deceleration rate lower than the 5.8 ft/s^2 described in the Green Book. The 5.8 ft/s^2 rate was approximately the 46th percentile value for vehicles at sites with lower speeds and taper rates, and it was the 33rd percentile value for vehicles at high-speed sites. The 50th percentile rate was approximately 6.1 ft/s^2 for low-speed sites and 6.7 ft/s^2 for high-speed sites. The 15th percentile deceleration rates at the high-speed and low-speed sites were approximately 4.2 ft/s^2 and 2.2 ft/s^2, respectively.

To better understand the relative effects of the site and vehicle variables on deceleration rate, researchers developed ANACOVA models that included many of the variables listed in Table 6-11. In reviewing the results of the various models, researchers observed that the statistical significance of the deceleration length changed based on whether all speed limits were considered together or whether they were divided into lower speeds (35-45 mph) and higher speeds (50 mph or higher). Further investigation revealed that taper length was confounded with posted speed limit; this is logical, considering that the design guidance recommends longer tapers for higher speeds. As a result, researchers focused their atten-

Table 6-16. Deceleration rates of left-turning vehicles prior to the end of the taper.

Site	Taper Rate[a]	Taper Length (ft)	Decel Length (ft)	Posted Speed Limit (mph)	Deceleration Rate Prior to End of Taper (ft/s^2)				Number of Vehicles
					Avg	Std Dev	15th %ile	85th %ile	
AL-09	6.8	61	94	35	11.7	4.9	2.9	14.8	17
FL-10	7.9	71	216	45	5.5	4.0	0.6	8.0	28
FL-09	8.0	80	173	35	3.9	2.2	1.4	7.0	33
MS-03	8.9	93	186	45	6.9	2.4	4.0	9.2	24
FL-03	9.3	98	365	45	7.8	3.8	2.5	12.3	38
AL-08	10.2	115	230	35	4.2	1.8	2.1	6.0	17
MS-08	11.9	119	255	45	5.7	3.2	1.7	9.2	59
MS-05	11.4	125	80	35	11.5	3.9	8.0	15.2	20
TX-33	13.2	152	312	65	5.9	3.6	2.0	8.3	43
AL-03	16.9	186	180	50	6.4	1.9	4.3	7.9	44
TX-28	16.6	191	283	65	8.1	2.5	5.8	10.3	55
TX-21	17.7	204	115	50	6.3	1.6	4.2	8.0	32
All Sites					6.7	3.6	2.8	9.8	410
All Low-Speed Sites (35–45 mph)					6.7	4.1	2.2	10.8	236
All High-Speed Sites (50–65 mph)					6.8	2.7	4.2	9.0	174

[a] Taper rate is the amount of longitudinal distance for each unit of transverse distance (i.e., the length of taper for each unit of lane width added).

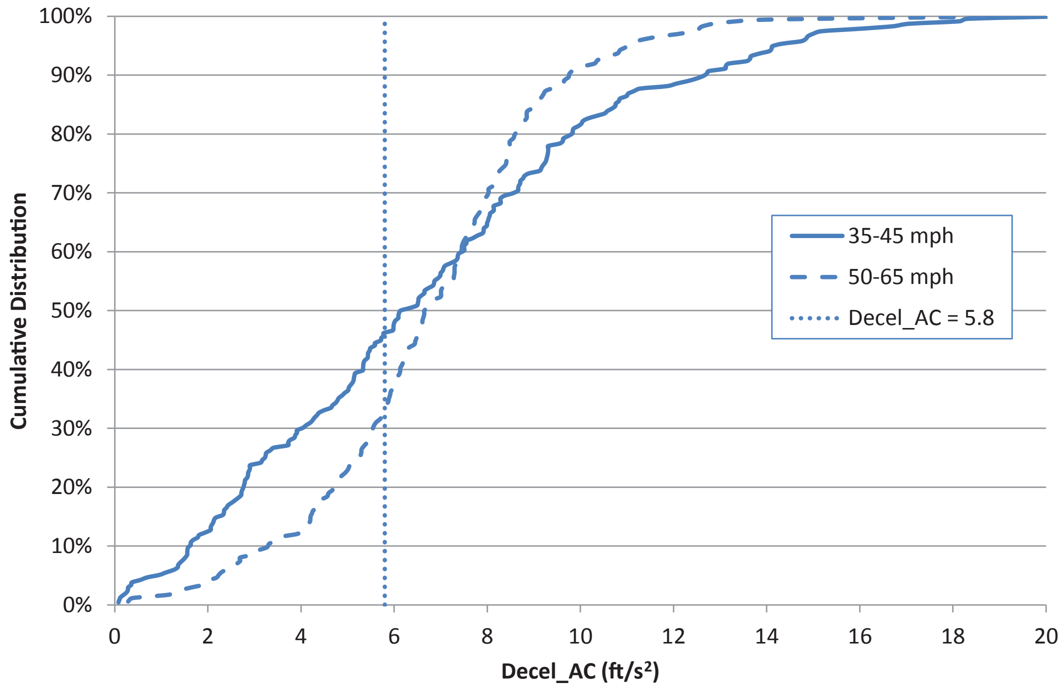


Figure 6-10. Cumulative distribution of deceleration rate prior to the end of the taper.

tion on the models that considered speed limits in separate categories. Those models are shown in Table 6-17 and 6-18. Those results show that the speed at the upstream counter and the speed in the taper were significant in affecting the rate of deceleration in the taper. The deceleration length was also significant at low-speed sites; indicating that some drivers consider all available length when completing their deceleration. The signs associated with the coefficients for each of these three variables are as expected. The signs for deceleration length and speed in the taper are negative, because as these are reduced, one expects to see an increase in deceleration rate. Conversely, deceleration rate through the end of the taper increases as the upstream speed increases.

While all three of these variables (Len_Decel, Spd_BC, and Spd_A) are statistically significant at $\alpha = 0.05$ for low-speed sites, the speed variables have coefficients with larger

Table 6-17. Evaluation of deceleration rate prior to the end of the taper (speed limit < 50 mph).

Response Decel_AC Speed_Group=Low (PSL_Major=35, 45)

Summary of Fit

RSquare	0.949283
RSquare Adj	0.948405
Root Mean Square Error	0.945365
Mean of Response	6.6861
Observations (or Sum Wgts)	236

Parameter Estimates

Term	Estimate	Std Error	DFDen	t Ratio	Prob>|t|
Intercept	0.1182101	2.278592	5.171	0.05	0.9606
Spd_A	0.7968167	0.015806	227.7	50.41	<.0001*
Len_Decel	-0.017306	0.00569	5.102	-3.04	0.0280*
Spd_BC	-0.591261	0.016227	230.3	-36.44	<.0001*
Len_Taper	-0.032898	0.021939	4.969	-1.50	0.1944

REML Variance Component Estimates

Random Effect	Var Ratio	Var Component	Std Error	95% Lower	95% Upper	Pct of Total
Site	1.9640842	1.7553315	1.142023	-0.483034	3.9936966	66.263
Residual		0.893715	0.0840813	0.7493649	1.0844395	33.737
Total		2.6490465				100.000

-2 LogLikelihood = 697.37297772

Table 6-18. Evaluation of deceleration rate prior to the end of the taper (speed limit ≥ 50 mph).

Response Decel_AC Speed_Group=High (PSL_Major=50, 65)

Summary of Fit

RSquare	0.901794
RSquare Adj	0.89947
Root Mean Square Error	0.857278
Mean of Response	6.800208
Observations (or Sum Wgts)	174

Parameter Estimates

Term	Estimate	Std Error	DFDen	t Ratio	Prob>|t|
Intercept	-2.539578	5.091504	1.009	-0.50	0.7048
Spd_A	0.4630709	0.013009	168.1	35.60	<.0001*
Len_Decel	0.0019478	0.005474	1.046	0.36	0.7803
Spd_BC	-0.299187	0.014917	168	-20.06	<.0001*
Len_Taper	-0.026365	0.022337	1.006	-1.18	0.4465

REML Variance Component Estimates

Random Effect	Var Ratio	Var Component	Std Error	95% Lower	95% Upper	Pct of Total
Site	0.3817164	0.2805331	0.4230742	-0.548692	1.1097586	27.626
Residual		0.7349256	0.080187	0.5999918	0.921376	72.374
Total		1.0154587				100.000

-2 LogLikelihood = 476.18761109

magnitudes. This is a function of the units in which the variables are reported; a 100-ft increase in deceleration length is associated with a decrease in deceleration rate of about 1.7 ft/s^2, while a 1-mph increase in upstream speed is associated with an increase in deceleration rate of about 0.8 ft/s^2. Thus, a 1-ft change in deceleration length does not have as great an effect on deceleration rate as a 1-mph change in speed, but deceleration length is important in predicting the deceleration rate at low-speed sites.

Results Compared to Green Book Guidance

The text in the Green Book (*2*) that describes taper design (pp. 9-127, 9-128) suggests the importance of both taper length and speed:

> On high-speed highways it is common practice to use a taper rate that is between 8:1 and 15:1 (longitudinal:transverse or L:T). Long tapers approximate the path drivers follow when entering an auxiliary lane from a high-speed through lane. However, with exceptionally long tapers some through drivers may tend to drift into the deceleration lane—especially when the taper is on a horizontal curve. Long tapers may constrain the lateral movement of a driver desiring to enter the auxiliary lanes. This situation 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 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 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.
>
> . . .
>
> 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 and greater.

The recommended taper rates and lengths in the Green Book are presented in conjunction with the speeds of the vehicles using the left-turn lane, so the design of the taper area leading into the full-width deceleration lane recognizes an influence of speeds, but the design guidelines in the text are not directly tied to the 5.8 ft/s^2 value presented in Table 9-22 (Table 6-2 in this document). Also, the guidelines give the designer the flexibility to use engineering judgment to determine the appropriate taper rate for design speeds between 30 and 50 mph.

Many of the study sites provided a taper length consistent with the recommended taper rates. If a taper rate of 15:1 is considered for left-turn lanes at sites with posted speed limits of 50 mph and higher, only TX-33 did not have at least 100% of the corresponding taper length; that site had 88% of the recommended length for that taper rate. Applying a taper rate of 8:1 for the remaining sites, only two sites (AL-09 and FL-10) did not have the corresponding length; AL-09 had 85% of the length needed for an 8:1 taper rate, and FL-10 provided 99 percent. Deceleration data from the study sites indicate that drivers are regularly using a deceleration rate that is higher than 5.8 ft/s^2 while navigating through the left-turn taper, even though many sites provided the recommended taper length.

To better understand the importance of this finding requires a more detailed inspection of specific components in the process that the Green Book uses to determine recommended lengths. The 5.8 ft/s^2 value in Table 9-22 is used in calculating what is called the "full deceleration length" in Figure 9-48 (Figure 6-1 in this document). That length includes both the taper (L_2) and the provided length within the full-width deceleration lane (L_3). A comparison of Figure 9-48 with Note 3 in Table 9-22 could suggest that the 5.8 ft/s^2 rate occurs solely within the taper, but Note 3 specifically states that the rate applies to the distance in which the turning vehicle moves from the through lane into the turn lane. That distance may or may not fully coincide with the taper length. Meanwhile, the recommended taper rate on pp. 9-127 and 9-128 (a value between 8:1 and 15:1) is given independently of the calculations in Table 9-22; the purpose of that taper rate is to provide a path into the left-turn lane that approximates the path that a driver would (or should) follow for a given speed during the lateral movement into the left-turn lane. The end result is that these two components of the left-turn taper design process are not directly linked, but the process provides a great deal of flexibility for the designer to consider site characteristics (e.g., speed and available right-of-way) to produce a left-turn lane design that meets the anticipated needs of a particular intersection.

Data from this study show that 85% of observed drivers at high-speed sites decelerated at a rate of 4.2 ft/s^2 or greater up to the end of the taper. A design that accommodates decelerating at 4.2 ft/s^2 during the lateral movement into the turning lane provides for a more gradual, controlled deceleration, but a higher deceleration rate (closer to 6.5 ft/s^2 for half of the observed drivers or 10 ft/s^2 for the most aggressive drivers) could be acceptable if site constraints or other factors dictate a shorter length. The tradeoff for the shorter length, however, would occur in one or both of the following forms:

- Less aggressive drivers would begin their deceleration earlier, either through coasting or applying the brake further upstream of the beginning of the taper, increasing the speed differential between turning and through vehicles.
- Some drivers would accomplish more of their deceleration after completing their lateral movement, leading to much higher deceleration rates approaching the stop line and/or the back of the queue.

Deceleration within the Deceleration Lane

A summary of deceleration rates within the full-width deceleration lane for each study site is shown in Table 6-19, with the sites ordered by deceleration length. The Green Book deceleration rate of 6.5 ft/s^2 is within the range of rates from the study sites, but 8 of the 12 sites had a higher average rate. In addition, the 15th percentile deceleration rate for all sites was 5.3 ft/s^2, and two of the 12 sites had a 15th percentile rate greater than 6.5 ft/s^2.

Figure 6-11 shows the cumulative distribution of the deceleration rates downstream of the taper (Decel_CE), in which about 35.6% of the observed left-turning vehicles had a deceleration rate lower than the 6.5 ft/s^2 described in the Green Book; the 6.5 ft/s^2 rate was approximately the 51st percentile value for vehicles at lower-speed sites and the 15th percentile value for vehicles at high-speed sites. The highest deceleration rate for both high-speed and low-speed sites was about 15 ft/s^2.

Results Compared to Green Book Guidance

Researchers considered that a possible explanation for the difference between the Green Book value of 6.5 ft/s^2 and the observed values is the fact that almost all of the study sites had deceleration lane lengths that were less than the Green Book's recommended values for those speeds. The Green Book (*2*) acknowledges the existence of this practice by saying that at

Table 6-19. Deceleration rates of left-turning vehicles in the deceleration lane.

Site	Taper Length (ft)	Decel Length (ft)	Posted Speed Limit (mph)	Deceleration Rate in Deceleration Lane (ft/s^2)				Number of Vehicles
				Avg	Std Dev	15th %ile	85th %ile	
MS-05	125	80	35	5.6	1.8	4.0	6.3	20
AL-09	61	94	35	5.0	1.2	3.7	6.1	17
TX-21	204	115	50	8.0	1.6	6.2	9.3	32
FL-09	80	173	35	7.4	1.7	5.0	9.3	33
AL-03	186	180	50	10.3	2.1	8.2	12.2	44
MS-03	93	186	45	8.1	2.5	5.6	9.8	24
FL-10	71	216	45	8.0	1.5	6.4	9.4	28
AL-08	115	230	35	6.0	1.5	3.8	7.7	17
MS-08	119	255	45	7.2	1.7	5.5	8.6	59
TX-28	191	283	65	7.8	1.9	5.7	10.0	55
TX-33	152	312	65	9.9	2.3	7.6	12.3	43
FL-03	98	365	45	5.3	1.4	4.0	6.5	38
All Sites				7.7	2.4	5.3	10.3	410
All Low-Speed Sites (35–45 mph)				6.7	2.0	4.6	8.8	236
All High-Speed Sites (50–65 mph)				9.0	2.3	6.5	11.4	174

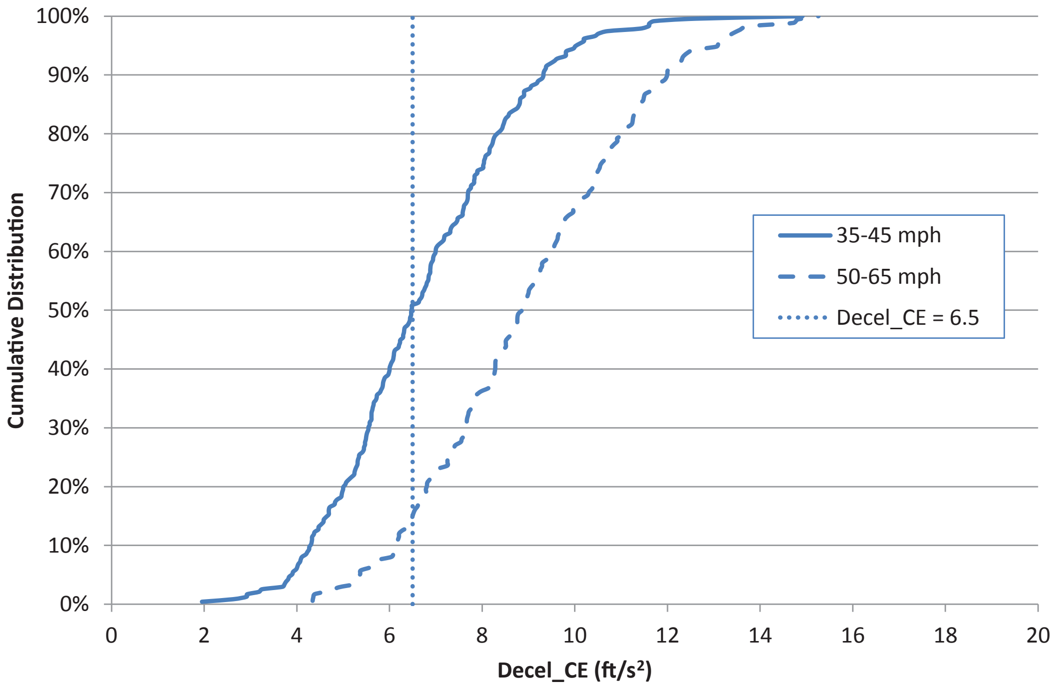


Figure 6-11. Cumulative distribution of deceleration rate within the deceleration lane.

many locations, "it is not practical to provide the full length of the auxiliary lane for deceleration due to constraints such as restricted right-of-way, distance available between adjacent intersections, and extreme storage needs," which is why there is an allowance for deceleration in the through lane upstream of the taper. (p. 9-127) The shorter lengths could persuade drivers that a more pronounced deceleration is needed to come to a stop at the stop line, especially if relatively little deceleration took place in the taper.

To further explore this theory, researchers developed ANACOVA models that would quantify the effects of deceleration length and other variables. Table 6-20 displays the

Table 6-20. Evaluation of deceleration rate in deceleration lane.

Response Decel_CE

Summary of Fit

RSquare	0.784076
RSquare Adj	0.780316
Root Mean Square Error	1.134194
Mean of Response	7.691544
Observations (or Sum Wgts)	410

Parameter Estimates

Term	Estimate	Std Error	DFDen	t Ratio	Prob>|t|
Intercept	-1.371806	5.304099	6.097	-0.26	0.8044
PSL_Major	0.0489858	0.053769	6.199	0.91	0.3963
Len_Taper	-0.001642	0.014231	6.112	-0.12	0.9119
Len_Decel	-0.01747	0.004671	6.251	-3.74	0.0089*
LW_Thru	-1.008134	0.426735	6.287	-2.36	0.0542
LW_LT	1.0671197	0.735762	6.14	1.45	0.1960
Spd_BC	0.3187896	0.013705	400.9	23.26	<.0001*
Spd_A	-0.005365	0.012702	398.1	-0.42	0.6730

REML Variance Component Estimates

Random Effect	Var Ratio	Var Component	Std Error	95% Lower	95% Upper	Pct of Total
Site	0.609386	0.7839113	0.4740314	-0.14519	1.7130129	37.865
Residual		1.2863953	0.0914063	1.1244849	1.486175	62.135
Total		2.0703065				100.000

-2 LogLikelihood = 1331.2432682

results of that analysis, in which deceleration length and the vehicle speed in the taper were found to be significant. Deceleration length is negatively related to deceleration rate, while speed is positively related; both of those relationships are as expected. As a result, the model suggests that, all other variables being held constant, a 10-ft increase in deceleration length will reduce the deceleration rate by about 0.2 ft/s^2, and a 1-mph increase in speed in the taper (i.e., at the start of the deceleration length) will increase the rate by approximately 0.3 ft/s^2.

Researchers also investigated whether the deceleration rate in the taper might have significantly affected deceleration in the full-width deceleration lane. The theory was that if vehicles decelerate more sharply within the taper that might lead to a decreased rate within the deceleration lane. A model similar to the one shown in Table 6-20 was run, substituting Decel_AC for Spd_A; however, Decel_AC was found to be not significant.

In the Green Book (*2*) text that discusses speed differential, the following statement is made on page 9-127:

> Shorter auxiliary lane lengths will increase the speed differential between turning vehicles and through traffic.

This relationship was evident in the speed differential data for the lower-speed sites in this study and inconclusive for the higher-speed sites. These findings along with the deceleration data findings support a conclusion that shorter auxiliary lane lengths increase the deceleration rate as turning vehicles approach the intersection. Compared to the 6.5 ft/s^2 rate noted in the Green Book, approximately two-thirds of the drivers observed making left turns at the study sites decelerated at greater rates to come to a stop at the stop line. Indeed, page 3-3 of the Green Book describes 11.2 ft/s^2 as a comfortable deceleration for most drivers in recommending a deceleration threshold for stopping sight distance, so rates greater than 6.5 ft/s^2 are not unique to this study. In terms of design guidelines, however, results from this study indicate that a designer could produce a left-turn lane design that is associated with a deceleration rate of 6.5 ft/s^2 and it would accommodate the current behavior of 85% of left-turning drivers at high-speed sites.

Suggested Changes to Green Book

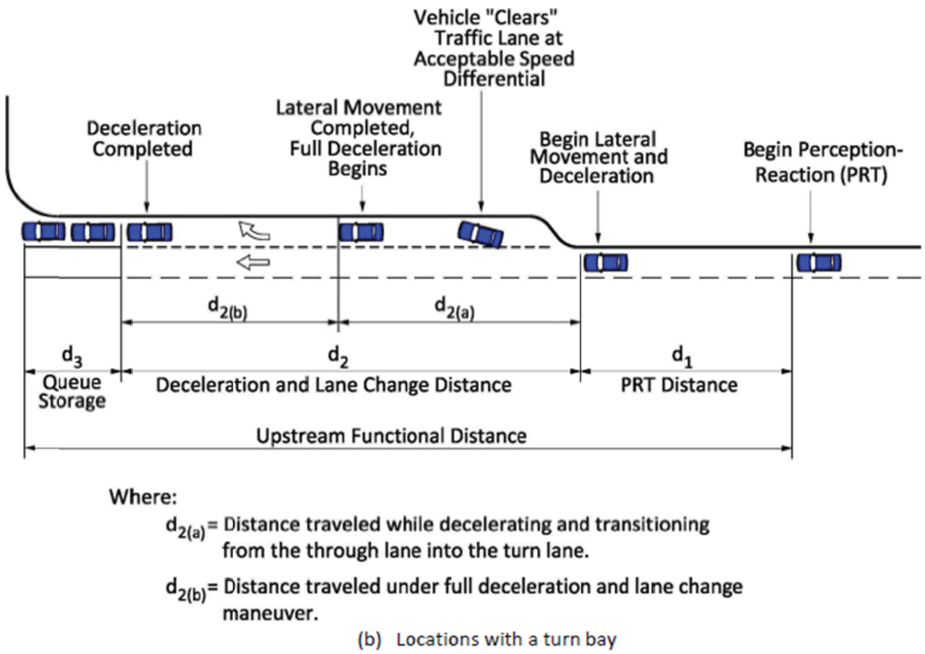
Based on the results of this study, a two-stage deceleration process that uses rates of 4.2 ft/s^2 during lateral movement into the turning lane and 6.5 ft/s^2 within the deceleration lane would accommodate most drivers. In locations with geometric constraints or high-speed approaches, a design that incorporates a higher deceleration rate could be offered as an alternative. Exhibit 14-4 of the draft second edition of the *Access Management Manual* (*97*) lists distances for "limiting conditions" of deceleration distance; while the manual states that these values are supported by the research conducted by Gates et al. (*95*), Chang et al. (*90*), and Williams (*99*), they are equivalent to a constant deceleration rate of 10 ft/s^2 throughout the full deceleration length (i.e., taper and full-width deceleration lane). In addition, the Gates, Chang, and Williams studies focused on drivers traveling straight through an intersection and responding to a change interval at a traffic signal.

While drivers may be able to negotiate the left-turn lane at higher deceleration rates, a design that accommodates lower rates provides a more conservative design that is less demanding on drivers and contains more provision for storage of queues of left-turning vehicles. That more conservative design could be accomplished through the use of a constant 6.5 ft/s^2 rate throughout the full deceleration length, which is very similar to the existing lengths in Table 9-22 of the Green Book. Substituting those values into Table 9-22, and including the intermediate speed values that are multiples of 5, would produce a result similar to Table 6-21.

Table 6-21. Deceleration lengths.

Speed, mph	Deceleration and Lane-Change Distance, ft[a]	
	Typical	Constrained
20	95	70
25	140	105
30	195	150
35	260	205
40	330	265
45	410	340
50	500	415
55	595	505
60	700	600
65	810	700
70	930	815

Notes:
1. The above full deceleration lengths are $d_{2a} + d_{2b}$ in Figure 6-10.
2. The speed differential between the turning vehicle and following through vehicles is 10 mph when the turning vehicle "clears the through traffic lane" (i.e., distanced_{2a} in Figure 6-10).
3. Deceleration lengths for a typical installation are based on 4.2 ft/s^2 deceleration while moving from the through lane into the turn lane (distance d_{2a}) and 6.5 ft/s^2 deceleration after completing the lateral shift into the turn lane (distanced_{2b}).
4. Deceleration lengths for a constrained installation are based on a 6.5 ft/s^2 deceleration throughout the entire length.
5. Typical lengths should be used for new roadway projects and for reconstruction projects where sufficient right-of-way exists to provide the additional length.
6. Deceleration rates are based on deceleration on dry, level pavement. Designs for approaches on downgrades of more than 2% and intersections at locations prone to wet pavement should account for the additional length necessary for vehicles to decelerate to a stop in those conditions.
7. Access points should not be allowed in the deceleration areas.
[a] Rounded to 5 ft.



Source: *Access Management Manual* (draft second edition), exhibit 14-2b. Reproduced with permission of the Transportation Research Board.

Figure 6-12. Upstream functional intersection area with a turn bay (97).

Figure 6-12 shows the second half of Exhibit 14-2 from the draft second edition of the *Access Management Manual* (*97*), illustrating the various key dimensions of a right-turn lane. Adapting this figure for use with left-turn lanes will help further illustrate the dimensions of Table 9-22 and relevant text in the Green Book. Figure 6-12 also shows that turning vehicles do not necessarily complete their lateral shift into the turning lane within the taper area. A shorter taper length, such as the 50-ft length used by the Florida Department of Transportation (*100*), provides an efficient way of adding the turning lane and allowing more length for the full-width deceleration lane.

CHAPTER 7

Conclusions, Recommendations, and Suggested Research

Project Summary

Researchers reviewed recent literature, state design manuals, and multiple editions of the AASHTO Green Book to determine the state of the practice and the basis for it. The research team also interviewed practitioners at state departments of transportation to gain additional insight into current design practices for deceleration lanes, multiple turn lanes, and channelizing island design. Using information from those activities, along with input from the project advisory panel, the research team conducted field studies on double left-turn lane operations and deceleration lane operations. Based on the findings from those field studies in conjunction with current practice and recent literature, the research team made recommendations for revisions to Chapter 9 of the 2011 AASHTO Green Book.

Literature Review

Researchers consulted various sources of literature, reviewing completed research findings (both recent and distant) and current research projects to determine the current state of the art and state of the practice on the design of auxiliary lanes. Results of the literature review, found in Chapter 2 of this document, focused on a selection of topics:

- Warrants (i.e., installation guidelines) for left-turn and right-turn lanes.
- Design of auxiliary lane components (i.e., deceleration, storage, and taper).
- Safety.
- Offset turn lanes.
- Intersection sight distance.
- Channelization.
- Effect of skew.
- Multiple turn lanes.
- Bypass lanes.
- Passing lanes.
- Alternative intersection designs.
- Acceleration lanes.
- Design tools.

State of the Practice—Review of Online Design Manuals

As part of Task 2 efforts, the research team conducted a state-of-the-practice review of current design considerations. A review of state online design manuals was conducted to identify (1) what is being discussed at the state level and (2) current design criteria being used for intersection auxiliary lanes. Table 2-10 summarizes the guidance provided in the state manuals.

State of the Practice—Discussions with Practitioners

To identify those practices and evaluations not adequately documented in traditional literature and/or design manuals, the research team contacted representatives from a selection of state departments of transportation to inquire about current practices and potential guidance needs, based on their professional experience and the policies of their respective departments. The information presented in Chapter 3 reflects the participants' responses as provided to the research team.

This state-of-the-practice questionnaire provided some insights into current issues related to the design of auxiliary lanes. The topics covered in the questionnaire were based on feedback from the project's advisory panel, and responses were received from 11 of the 12 state DOTs contacted. Following are findings from the responses received.

Deceleration Length

- There are multiple ways to define a deceleration lane. When discussing deceleration lane design, care should be taken to ensure that all participants are using the same definition.

- Reasons for installing a deceleration lane and/or determining its length are typically based on various other geometric and traffic considerations, capacity, and speed being among the most common.
- Agencies use numerous sources as the basis for their deceleration lane guidelines. The AASHTO Green Book and *NCHRP Report 279* were the most common and have similar roots in Harmelink's work, but other sources are also used, including state-specific manuals and other national documents.
- When the preferred dimensions of a deceleration lane cannot be accommodated within a particular site, the decision for how to make adjustments and reductions is typically a qualitative one, though the factors that contribute to that decision are not always well defined.
- Respondents were generally satisfied with existing guidance, though there were suggestions to include better methods of projecting future turning volumes for determining storage and better guidance on the type of widening or shadowing that is appropriate for a given auxiliary lane.

Multiple Turn Lanes

- Double Left-Turn Lanes:
  - Most agencies have some kind of guidance regarding double left-turn lanes, though it may not be very detailed.
  - The decision to install such a treatment is frequently based on the current or expected turning demand, but signalization is also a typical criterion.
  - Guidance on the design of receiving lanes is often described as the width of the turning curve, though it may also be described in terms of receiving lane width.
  - The capacity of a double left-turn lane was generally viewed to be less than twice that of a single lane, but the exact value of that capacity was not universally agreed upon.
  - Guidance on how to adjust the three components of a double left-turn lane was widely varied, if available. The adjustments for storage, deceleration, and taper of a double left-turn lane were often determined qualitatively or on a case-by-case basis.
  - Some of those issues were reflected in respondents' suggestions for added guidance, desiring information on lengths for given design speeds or for urban vs. rural settings.
- Triple Left-Turn Lanes:
  - Existing guidance on triple left-turn lanes is very limited, and in some states the treatment is heavily discouraged or prohibited.
  - The guidance that does exist is typically similar to that found for double left-turn lanes, with additional considerations for the turning curve and receiving lanes.
- Double Right-Turn Lanes:
  - Existing guidance for double right-turn lanes is less common than that for double left-turn lanes but more common than triple left-turn lanes.
  - Design guidance for a double right-turn lane is frequently similar, if not identical, to guidelines for designing double left-turn lanes, though site-by-site analysis is also important.
  - Several respondents desired additional information, such as consideration of pedestrians, appropriate lengths for given design speeds, and warrants for installation.

Island Design for Right-Turn Lanes

- Guidance on island design is found in many states, though that guidance is not always very detailed.
- Some states' guidelines contained recommendations for island sizes, approach angles, turning volumes, and pedestrian provisions, but many materials were more general in nature.
- Preferences for flat-angle entry to the cross street were slightly higher than preferences for entry closer to a right angle.
- Various traffic and geometric characteristics contribute to the design of a turning island, including traffic volume, intersection size and angle, pedestrian accommodation, sight distance, and speed.
- Little additional information was suggested for inclusion in guidance documents that is not currently provided, although there was a concern for pedestrian accommodation when an island is installed.
- Respondents suggested that some island designs could be improved with further consideration of large vehicles, accommodating pedestrians and older road users, approach angle and taper, and sight distance.

Typical Designs

As part of the questionnaire sent to key state transportation agencies, respondents were asked to identify locations with installations that would be considered best-practice sites. These best-practice sites were to demonstrate preferred design treatments for five design categories: island design, deceleration lane design, double left-turn lane design, triple left-turn lane design, and double right-turn lane design. Colorado, Florida, Maine, Minnesota, North Carolina, and Washington all identified locations for consideration.

An underlying question associated with the identification of these locations centered on whether the Green Book provides sufficient guidance for implementing the treatment.

Each of the identified sites was examined using aerial imagery to gain a better understanding of the key features of the design, as implemented. A single site considered representative of the design treatment was identified for each of the five design categories. This representative site was examined in detail through a case study approach.

The case studies focused on the key design features associated with the treatment under consideration. To the extent possible, these design features were quantified and referenced to the appropriate standards. A review of each of the completed case studies was then done to determine if the guidance provided by the Green Book would have been sufficient or if supplemental guidance would have been necessary to complete the design. In those cases where supplemental guidance was necessary, this supplementary guidance has been summarized.

The evaluation of best-practice sites that demonstrate preferred design treatments for five design categories—island design, deceleration lane design, double left-turn lane design, triple left-turn lane design, and double right-turn lane design—show that the guidance found in the 2011 Green Book is adequate for island design. However, supplemental guidance may be needed to address adequately the design needs of providing deceleration lanes in a flared intersection configuration, which occurs when adding a turn lane for an undivided roadway, and to address clearance distances for multiple turn lanes.

Preliminary Review of Issues

As part of the initial tasks for this project, the research team identified a list of issues to consider for further study. This list was provided to the panel and discussed during a conference call. At the conclusion of the conference call, the following issues were prioritized for potential Phase II studies:

- Multiple turn lanes.
- Deceleration lane (validation of components, safety/operations of different lengths).
- Island design/channelization for right-turn lanes.
- Pedestrian/bicycle/complete street issues.

Field Studies and Final Green Book Review

In Task 3 of the project, the research team developed an interim report that described the activities and findings from Tasks 1 and 2. That interim report was accompanied by a list of possible research topics for review by the project advisory panel; in accordance with previous direction from the panel, possible research topics were focused on aspects of multiple left-turn lanes and left-turn deceleration lanes.

The research team and the panel met to discuss the findings from Tasks 1 and 2 to determine the emphases of the field studies in Phase II of the project. The panel selected two field studies for completion in Phase II:

- Operational study on double left-turn lanes.
- Operational study on deceleration lanes.

Both studies were chosen with the intent that they would produce recommendations for revisions to the Green Book, and those recommendations are included with other suggested changes identified in the final task of the project, the Green Book review.

Conclusions

Double Left-Turn Lane Operations

The goal of the double left-turn lane operational study was to determine the effects of geometric characteristics on double left-turn lane operations, as measured with saturation flow rate, lane distribution, and driver behaviors. Receiving leg width, left-turn lane width, and downstream friction type and distance were the key geometric variables studied. Identifying sites with the desired range of receiving leg width was the most difficult of the study variables to satisfy, although finding sites with an average double left-turn lane width greater than 12 ft also proved challenging. Data from 26 sites in three states (Arizona, California, and Texas) were used in the analyses. The data collection method was video recording.

The ITE *Manual of Transportation Engineering Studies* (*79*) procedure guided the determination of the saturation flow rate. The procedure requires that each cycle must have more than seven vehicles and only passenger cars in the traffic stream are to be considered in the determination of saturation flow rate. Because of challenges with obtaining sufficient sample size, queues five to 10 vehicles in length were included in the analysis. The number of vehicles in the queue was a variable added to the model in case the shorter queues had a different saturation flow rate than the longer queues. The time each left-turning vehicle crossed the stop bar was logged, and these times were used to determine the headway between following vehicles. Also recorded was whether the vehicle was a truck as well as whether the vehicle was not in the queue at the start of the cycle. If either case was true, then the queue was eliminated from the study. The times between vehicles were used to calculate the saturation flow rate.

Saturation flow rate represented the number of vehicles served by one lane over 1 hr of green time. It was calculated using the headway between following vehicles when all vehicles being considered were passenger cars and were present at the start of the green phase. The headways for the first four vehicles were dropped from the calculation. A total of 10,023 saturation flow rate values were available for study. The average double left-turn lane saturation flow rate for these 10,023 data points was 1775 pcphgpl.

At some DLTL intersections, drivers may choose one left-turn lane over another left-turn lane in anticipation of a turn at a downstream intersection or because of familiarity with a downstream friction point. The data available in this study were used to investigate if geometric elements are associated with how drivers distribute within the double left-turn lanes. A variable called lane share was created to calculate the percentage of the volume present within a cycle to each lane. If the queues were equal between the lanes, the lane share variable for Lane 1 was 50% and 50% for Lane 2. This lane share variable was aimed at determining the proportion of vehicles that use a lane out of all left-turn vehicles recorded during a cycle in order to provide a good measure of the distribution of left-turn demand of passenger cars across the double left-turn lanes.

Driver behavior may be related to the geometric design characteristics of the double left-turn lanes, which could affect the saturation flow rate (i.e., operations) or the safety of the intersection. For example, slow-to-start behaviors may decrease the saturation flow rate and increase the potential for rear-end crashes. Driver behaviors of interest for double left-turn lanes were identified from the literature in conjunction with the engineering judgment of the research team. Technicians then watched the video and documented whenever one of those driver behaviors occurred.

Key findings from the analyses of operations at double left-turn lanes include the following:

- The lane variable was found to be not significant, which means that the inside and outside lane saturation flow rates were similar.
- The number of vehicles in the queue was also not significant. In other words, whether the queue length was five vehicles or ten vehicles, similar saturation flow rates were measured after controlling for variations caused by other variables within the model.
- Because U-turns require drivers to slow more than they would for a left turn, it is reasonable to assume that a U-turn will also take more time and therefore negatively affect saturation flow rate. The model found that for each additional U-turning vehicle within the left-turn queue, saturation flow rate decreased by 56 pcphgpl.
- The analysis of the effects of the friction point type and location revealed that the analysis needed to include a new variable. The new variable accounted for a dedicated lane added at the end of a channelized right turn. While the turning vehicles were constrained to two lanes at the start of the receiving leg, a review of the video data revealed that drivers in the outside lane would angle their vehicle to make a smooth entry into the new lane. This behavior resulted in higher saturation flow rates, as demonstrated with the variable being significant. The model results indicated that the addition of this new lane resulted in an increase in saturation flow rate of about 50 pcphgpl.
- The *Highway Capacity Manual* (*8*) indicates that wider lane widths are associated with higher saturation flow rate. One of the findings from this DLTL study was that the width of the left-turn lanes did not significantly affect the saturation flow rate. This finding could be construed to mean that narrow lanes can be used without affecting operations. In making this interpretation, however, a key component of the study design is not represented. The recommended method to determine saturation flow rate requires the elimination of a queue if a heavy vehicle is present within the queue. Therefore, within this study, while the operations of queues with only passenger cars were similar for the various left-turn lane widths studied (9.5 to 13 ft), the operations of queues that include heavy vehicles (trucks or buses) may have different results.
- The width of the receiving leg represents the visual target for the left-turning drivers. For the sites included in this study, the width of the receiving leg ranged from 24 ft to 54 ft. The analysis did find that the width of the receiving leg affected the saturation flow rate. While significant, the incremental difference in saturation flow rate for an incremental increase in leg width was small. The pattern of increasing saturation flow rate for increasing receiving leg width was examined to try to identify if there were dimensions where a sizable increase in saturation flow rate occurs. The change point detection analysis based on predicted saturation flow rates identified a receiving leg width of 36 ft as the change point. When the receiving leg width was between 24 and 36 ft, the average saturation flow rate was 1725 pcphgpl, while a receiving leg width of 40 to 54 ft was associated with an average saturation flow rate of 1833 pcphgpl. Determining whether the benefits of the additional saturation flow outweigh the costs (e.g., maintenance, construction, and/or right-of-way) was not a component in this study; therefore, the benefit-cost ratio may be very limited.
- As demand increases, the selection of which left-turn lane to enter may be more of a reflection of drivers selecting the lane with the shorter queue rather than being concerned with downstream conditions. Therefore, the evaluation on lane distribution only used the queues in which fewer than 11 vehicles were present within the queue. Several variables were found to be significant; however, almost all of the prediction was accomplished by the Lane 1 and the Lane 2 volumes, indicating that drivers are making lane selection based on the length of queues present when the driver approaches the intersection.
- For the 18 sites included in the driver behavior review, the most common behavior was lane changes before the friction location, followed by lane changes at the friction location. As

expected, many sites with driveways/intersections as the friction point were associated with high numbers of lanes changes. The behaviors that reflected conflicts or potential conflicts were reviewed to identify if there were common elements. The three sites with the largest number of these types of behaviors all had a friction point within 150 ft of the intersection.

Deceleration Lane Operations

The goal of the deceleration lane study was to determine the effects of taper length and posted speed limit on approach speeds and deceleration rates of left-turning vehicles, as compared to those described in the Green Book. Deceleration length, through lane width, left-turn lane width, and operating speeds at key points along the approach through the left-turn lane were also measured and included in the analysis. Identifying sites with medians that contained a distribution of posted speed limits and taper lengths was the most challenging part of site selection, though the research team also sought to identify sites where proximity improved efficiency of the data collection effort. Data from 12 sites in four states (Alabama, Florida, Mississippi, and Texas) were used in the analyses. The primary data collection method was video recording.

Researchers observed and recorded images of vehicles that completed left turns at signalized intersections with medians. The recordings were used to document the times at which each vehicle reached the beginning of the left-turn taper, the end of the taper, a predetermined point within the deceleration lane, and the stop line. Using the elapsed time to travel between those locations and the corresponding distances, researchers calculated speeds and deceleration rates of the left-turning vehicles. To verify these calculations, researchers also collected limited amounts of data with lidar and with GPS, comparing the data from those efforts to that of the video. Researchers also used an automated traffic counter to collect spot-speed data upstream of the left-turn lane to provide an indication of the approach speeds of left-turning vehicles as well as likely speeds of through vehicles as they continued through the intersection. Following data reduction and quality control checks, data from 410 left-turning vehicles were analyzed to investigate three key deceleration lane design guidelines in the Green Book:

- The speed differential between turning vehicles and following through vehicles is 10 mph when the turning vehicle clears the through-traffic lane (see Note 3 in Table 6-2).
- The values for deceleration length are based on a 5.8 ft/s^2 average deceleration while moving from the through lane into the left-turn lane (see Note 4 in Table 6-2).
- The values for deceleration length are based on a 6.5 ft/s^2 average deceleration after completing the lateral shift into the left-turn lane (see Note 4 in Table 6-2).

Key findings from the analyses of operations at deceleration lanes were:

- The Green Book currently states that a 10-mph differential is acceptable on arterials, but drivers commonly had speed differentials greater than 10 mph in this study. Furthermore, the higher speed differentials in this study occurred for vehicles traveling at higher speeds. The Green Book describes the 10-mph differential in relation to a comfortable deceleration for the driver, while support for using 10 mph as the threshold for speed differential may be better stated in terms of reducing the likelihood of a crash.
- The speed differential was significantly and positively related to the upstream speed. Other variables, such as posted speed limit, deceleration length, and taper length also had various effects on speed differential in the statistical models, and the general effect of deceleration length appears to be consistent with the Green Book text that describes higher differentials with shorter length, but those effects were not statistically significant at the 0.05 level.
- The Green Book deceleration rate of 5.8 ft/s^2 prior to the end of the taper was within the range of average rates at the study sites, but eight of the 12 sites had an average rate higher than 5.8 ft/s^2. The 50th percentile rate was approximately 6.1 ft/s^2 for low-speed sites and 6.7 ft/s^2 for high-speed sites. In addition, 85% of observed drivers at high-speed sites decelerated at a rate of 4.2 ft/s^2 or greater up to the end of the taper.
- The deceleration length (at low-speed sites), the speed at the upstream counter, and the speed in the taper were all significant in affecting the rate of deceleration prior to the end of the taper. Coefficients for deceleration length and speed in the taper have negative signs because as these variables are reduced, one expects to see an increase in deceleration rate. Conversely, deceleration through the end of the taper increases as the upstream speed increases.
- The recommended taper rates and lengths in the Green Book are presented in conjunction with the speeds of the vehicles using the left-turn lane, so the design of the taper area leading into the full-width deceleration lane recognizes an influence of speeds, but the design guidelines in the text are not directly tied to the 5.8 ft/s^2 value presented in Table 9-22. Many of the study sites provided a taper length consistent with the recommended taper rates, but data from this study show that drivers commonly traveled through the taper at deceleration rates higher than 5.8 ft/s^2 for taper rates between 6:1 and 18:1.
- A design that accommodates decelerating at 4.2 ft/s^2 during the lateral movement into the turning lane provides for a more gradual, controlled deceleration, but a higher deceleration rate (closer to 6.5 ft/s^2 for half of the observed

drivers or 10 ft/s^2 for the most aggressive drivers) could be acceptable if site constraints or other factors dictate a shorter length. The tradeoff for the shorter length, however, would occur in one or both of the following forms:
  - Less aggressive drivers would begin their deceleration earlier, either through coasting or applying the brake further upstream of the beginning of the taper, increasing the speed differential between turning and through vehicles.
  - Some drivers would accomplish more of their deceleration after lateral movement, leading to much higher deceleration rates approaching the stop line and/or the back of the queue.
- Deceleration length and the vehicle speed in the taper were found to be statistically significant in determining deceleration rate within the full-width deceleration lane. Deceleration length was negatively related to deceleration rate (i.e., shorter lengths had higher deceleration), while speed was positively related; both of those relationships are as expected.
- Compared to the 6.5 ft/s^2 rate noted in the Green Book, approximately two-thirds of the drivers observed making left turns at the study sites decelerated at greater rates to come to a stop at the stop line. Data from this study indicate that a designer could produce a left-turn lane design that is associated with a deceleration rate of 6.5 ft/s^2 and it would accommodate the current behavior of 85% of left-turning drivers at high-speed sites and half of drivers at low-speed sites.
- In the Gates et al. study (*95*), deceleration was measured for drivers going straight. In this study, deceleration was measured for left-turning drivers while changing lanes in preparation for a stop at the stop line. This study found slightly slower deceleration rates as compared to the Gates et al. study. The deceleration rates for vehicles going straight on roads above 40 mph were 9.2 ft/s^2 (15th percentile), 10.9 ft/s^2 (50th percentile), and 13.6 ft/s^2 (85th percentile). The deceleration rates for left-turning passenger cars that changed lanes on roads with posted speed limit greater than 45 mph were 6.5 ft/s^2 (15th percentile), 8.9 ft/s^2 (50th percentile), and 11.4 ft/s^2 (85th percentile).

Final Green Book Review

A complete description of all recommended improvements to Chapter 9 of the 2011 Green Book is provided in Appendix A of this report. This section summarizes the changes identified by the research team as a result of the activities on this project.

Section 9.3, Introduction (pages 9-8 to 9-10)

- Recommended minor changes in text, including replacing the term "free-flow right-turn lanes" with "channelized right-turn lanes" to correspond with results from NCHRP Project 03-89 (*28*).

Section 9.3.1, Three-Leg Intersections (pages 9-10 to 9-14)

- Recommended expanded discussion of bypass lanes, including a cross-reference to warrants suggested for inclusion in Section 9.7.3, based on research in NCHRP Report 745 (*9*).
- Recommended revisions to some existing diagrams to improve legibility, provide additional detail, and add conflict diagrams.

Section 9.3.2, Four-Leg Intersections (pages 9-14 to 9-19)

- Recommended additional cross-references to guidance on auxiliary lane design in Section 9.7 for improved clarity.
- Recommended updating the text to better reflect current practice and guidance (including the *Highway Design Handbook for Older Drivers and Pedestrians*) on skew angle, which should not be less than 75 degrees.
- Recommended revisions to some existing diagrams to improve legibility, provide additional detail, and add conflict diagrams.

Section 9.4.2, Alignment (page 9-27)

- Recommended updating the text to better reflect current practice and guidance (including the *Highway Design Handbook for Older Drivers and Pedestrians*) on skew angle, which should not be less than 75 degrees.

Section 9.4.3, Profile (page 9-27)

- Recommended updating the text to include specific references to federal accessibility guidelines.

Section 9.6.1, Types of Turning Roadways (pages 9-55 to 9-88)

- Recommended inserting a new subsection on page 9-55 on channelized right-turn lanes, to improve the level of detail currently found in the Green Book, based on research in NCHRP Project 3-89 (*28*).
- Recommended a revision to the text on page 9-80 to include environmental issues as a factor in justifying tradeoffs in design, as recommended by NCHRP Synthesis 422 (*101*).
- Recommended a change in the description of curb radii and refuge islands that accompanies Green Book Figures 9-34 and 9-35 on page 9-88. References are provided to guidelines from AASHTO, ITE, and Virginia DOT.

Section 9.6.2, Channelization (pages 9-93 to 9-94)

- Recommended addition to the text on page 9-93 to promote the use of median storage for vehicles turning left from a minor road onto a divided highway.
- Recommended updating the text to better reflect current practice and guidance (including the *Highway Design Handbook for Older Drivers and Pedestrians*) on skew angle, which should not be less than 75 degrees.
- Recommended the use of "storage" instead of "refuge" to describe the storage distance for turning vehicles, to prevent confusion with refuge areas for pedestrians or bicyclists.

Section 9.6.3, Islands (pages 9-97 to 9-104)

- Recommended a revision of Green Book Figure 9-37 to improve legibility and correct perceived alignment errors.
- Recommended adding text to describe the appropriate length of the transition taper as the value calculated by Equation 3-37 or 3-38 on page 3-134. Most state design manuals use a form of those taper equations, and the addition would provide clarification as well as consistency with current practice.

Section 9.6.4, Free-Flow Turning Roadways at Intersections (page 9-106)

- Recommended removal of this section, including Figure 9-42. This information deals more with the exit design from either a freeway or grade-separated arterial, which is already covered in Chapter 10 (Section 10.9.6 Ramps). Further, it confuses the issue with relation to corner islands.

Section 9.7.1, General Design Considerations (pages 9-124 to 9-125)

- Recommended revision to the text to clarify the description of appropriate use of acceleration lanes.

Section 9.7.2, Deceleration Lanes (pages 9-125 to 9-130)

- Recommended revision of the section on deceleration length on pages 9-125 to 9-127 to incorporate the findings from this project's Task 4 deceleration field study. This included adding a subsection on perception-reaction distance. It also included replacing Figure 9-48 and Table 9-22 for clarity and consistency.
- Recommended revision of the section on taper length on pages 9-127 to 9-130 to better define the purpose, dimensions, and design of a taper used to add a left-turn lane. This revision also incorporates the findings from this project's Task 4 deceleration field study.

Section 9.7.3, Design Treatments for Left-Turn Maneuvers (pages 9-131 to 9-139)

- Recommended changing the structure of the section beginning on page 9-131 to better reflect its content. Much of the subsection entitled Guidelines for Design of Left-Turn Lanes really focuses on installation of left-turn lanes. This subsection could be split into multiple parts, to include discussion of installation, warrants, and general design principles. The existing text in the first two paragraphs is on installation. The suggested revised text is presented as a subsection on warrants for left-turn lanes and bypass lanes, based on research in NCHRP Report 745 (*9*). The remaining text can then be the Guidelines for Design of Left-Turn Lanes subsection.
- Recommended revising the text in the section on page 9-139 to better describe the capacity benefits and design considerations for multiple left-turn lanes, based on findings from the double left-turn lane operational study in Task 4 of this project.

Section 9.8.1, General Design Considerations (page 9-140)

- Recommended revising this section to include discussion on the value and benefits of directional and bidirectional crossovers, modification of median openings, and vehicle storage in wide medians. There is currently no guidance listed for under which conditions a median opening should be eliminated or when it should be made directional. This is compounded by the fact that there is no discussion of the difference between a bidirectional (conventional) crossover and a directional crossover. Additional sources for information on median openings are in the following: Michigan DOT *Geometric Design Guide 670* (*102*), Michigan DOT *Road Design Manual* (*103*), FHWA *Alternative Intersections/Interchanges: Informational Report* (*104*), and NCHRP Report 650 (*105*). There is no discussion of the impact of providing markings or a divisional island for vehicle storage in the median. Guidance regarding the impact of proper pavement markings on driver behavior should be added as available in NCHRP Report 650 (*105*).

Section 9.8.2, Control Radii for Minimum Turning Paths (page 9-144)

- Recommended adding text to emphasize the safety impacts of minimizing the median opening length. Much of the discussion focuses on curve radii for the median design but forgets that the overall length of the opening is important as well. NCHRP Report 650 provides discussion regarding median design (*105*).

Section 9.8.3, Minimum Length of Median Opening (page 9-149)

- Recommended revising text for clarity and to provide a reference to supporting research.

Section 9.9.1, General Design Considerations (pages 9-155 to 9-157)

- No changes were specifically recommended, but researchers recommended including a placeholder for a guidebook scheduled to be completed in an ongoing research project. The placeholder was added as a reminder to review this resource when it becomes available and determine if it should be added as a reference resource to this section.

Section 9.9.2, Intersections with Jughandle or Loop Roadways (pages 9-157 to 9-160)

- Recommended additional text and a table to provide guidance on the spacing of jughandle intersections and the corresponding primary intersection.
- Recommended revising Figures 9-60 and 9-62 for clarity and to provide a conflict diagram.
- Recommended text revisions to remove inconsistencies in terminology and to better describe operations at the minor intersections.
- Recommended adding a reference to the *Alternative Intersections/Interchanges: Informational Report* (*104*).

Section 9.9.3, Displaced Left-Turn Intersections (pages 9-160 to 9-161)

- Recommended additional text and a table to compare conflict points for continuous-flow intersections and traditional intersections.
- Recommended adding a reference to the *Alternative Intersections/Interchanges: Informational Report* (*104*).

Section 9.9.4, Wide Medians with U-Turn Crossover Roadways (pages 9-162 to 9-164)

- Recommended additional text and a replacement for Figure 9-65 to better discuss the restricted crossing U-turn intersection.
- Recommended adding a comment about using loons to accommodate larger vehicle U-turns with narrow medians.
- Recommended adding text, tables, and figures to better describe the median U-turn intersection and the restricted crossing U-turn intersection.
- Recommended adding a reference to the *Alternative Intersections/Interchanges: Informational Report* (*104*).

Section 9.9.5, Location and Design of U-Turn Median Openings (pages 9-165 to 9-166)

- Recommended revisions to the text and Table 9-30 to correct inconsistencies in the values for median widths.
- Recommended updating the text on wide medians and adding a new figure to show a typical loon design in order to better represent current practice.
- Recommended revising text to describe the location of midblock median U-turn openings to improve consistency with current practice.
- Recommended revising text and adding figures to bring description of multiple U-turn lanes into agreement with the state of the practice.

Section 9.10, Introduction (pages 9-167 to 9-169)

- Recommended adding a paragraph to the end of the section to emphasize the benefits of public outreach and education.

Section 9.10.2, Fundamental Principles (pages 9-173 to 9-175)

- Recommended adding a paragraph on right-turn bypass lanes to the end of the section entitled “Lane Balance and Continuity” to fill an information gap.
- Recommended adding a paragraph on turbo roundabouts to the end of the section entitled “Appropriate Natural Path” to fill an information gap.
- Recommended adding a paragraph on large, oversized, and superload vehicles to the end of the section entitled “Design Vehicle” to fill an information gap.
- Recommended adding a reference to NCHRP Report 674 (*65*).

Recommendations

The following sections present the recommendations from the field studies conducted in this project.

Double Left-Turn Lane Operations

Researchers used the findings from this study to develop recommendations on geometric design features that affect double left-turn lane performance. Potential recommendations include the following:

- The Green Book states that the capacity of double left-turn lanes is approximately 180% of that of a single median lane. Per the *Highway Capacity Manual* (*8*), the base saturation flow rate for a metropolitan area with population of 250,000 is 1900 pcphgpl and the left-turn adjustment factor is 1/1.05. Comparing the single-lane saturation flow rate (1900/1.05 = 1810 pcphgpl) to the average saturation flow rates for the double left-turn lanes sites in this study (1774 + 1776 = 3550 pcphgpl) results in a value (3550/1810 = 1.96 or 196 percent) that is greater than 180 percent.

- The Green Book states that the receiving leg of the intersection should have adequate width to accommodate two lanes of turning traffic and that a width of 30 ft is used by several highway agencies. Early literature by Neuman (*5*) stated the throat width for the turning traffic is the most important design element, and that because of the offtracking characteristics of vehicles, a 36-ft throat width is desirable for acceptance of two lanes of turning traffic. In constrained situations, 30-ft throat widths are acceptable minimums. Within this study, the pattern of increasing saturation flow rate for increasing receiving leg width was examined to try to identify if there were dimensions where a sizable increase in saturation flow rate occurs. The method concluded that the change point occurs between receiving leg widths of 36 ft and 40 ft.
- Additional discussions or cautions in the section on multiple turn lanes:
  – Double left-turn vehicles, turning into a receiving leg of two lanes where a third lane was being added as a dedicated downstream lane from a channelized right-turn lane, were observed to move into the additional lane as soon as physically possible, even across a solid white line.
  – The number of U-turning vehicles has a significant impact on the operations of double left-turn lanes.

Deceleration Lane Operations

Researchers used the findings from this study to develop recommendations on geometric design features that affect deceleration lane performance. Potential recommendations include the following:

- The Green Book states that a 10-mph differential between turning and through vehicles is commonly considered acceptable on arterial roadways, though 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. Given that higher speed differentials also occur on high-speed arterials as found in this research, it may be useful for the Green Book to clarify on what basis that differential is "acceptable" and add the references to the research or other guidance documents that support the explanation. For example, referencing research by Soloman (*98*) or others that describe the increased likelihood of crashes as speed differential increases above 10 mph establishes a basis for recommending that designers provide left-turn lane designs that do not require speed differentials greater than 10 mph. This research also identified smaller speed differences for sites with longer combined taper and deceleration lengths.
- The support for selecting the deceleration values of 5.8 ft/s^2 and 6.5 ft/s^2 noted in Table 9-22 of the Green Book is not directly referenced in the table or described in the guidelines in the accompanying text. Drivers are able to comfortably decelerate at greater rates, though a design that accommodates lower rates provides a more conservative design that is less demanding on drivers and contains more provision for storage of queues. However, if that conservative design is the desired result, the guidelines in the Green Book should contain additional text and references to relevant research supporting the use of those values. Based on the results of this study, a two-stage deceleration process that uses rates of 4.2 ft/s^2 during lateral movement into the turning lane and 6.5 ft/s^2 within the deceleration lane would accommodate most drivers. In constrained locations, a single deceleration rate of 6.5 ft/s^2 over the full deceleration length would suffice. Providing designs that require greater deceleration rates would likely result in larger speed differences between turning and through vehicles as turning drivers choose to complete more of their deceleration upstream of the taper.

Review of Green Book

Researchers completed two reviews of Chapter 9 (Intersections) of the 2011 AASHTO Green Book. Comparing the existing guidance in Chapter 9 with results from this research project as well as recent research and current practice, the research team compiled a list of recommended revisions to text, figures, and tables. Specifically related to the findings from this research project, researchers recommended that the Green Book guidance be revised to reflect new recommendations for:

- Typical and constrained deceleration lengths at left-turn lanes.
- Deceleration rates associated with recommended deceleration lengths.
- Text to better describe the capacity benefits and design considerations for multiple left-turn lanes.

Suggested Research

Based on the findings from the literature review, state-of-the-practice review, and field studies on this project, researchers suggest the following topics to be considered for further research:

- The safety performance of double left-turn lanes (i.e., where crashes occur and how they can be prevented).

This project studied operational characteristics and their relationship to design elements. A similar study on what design characteristics (e.g., lane width on approach, receiving lane/leg width, separation between opposing left-turn lanes, presence of pavement markings, location of downstream friction points) affect the type (e.g., sideswipe, rear-end) and frequency of crashes could be equally beneficial.

- The characteristics of lane-change maneuvers into the left-turn lane (i.e., where maneuvers occur relative to the beginning and end of the taper). Results from the deceleration study in this project support other findings and guidance that indicate the lane-change maneuver is not strictly confined to the taper area. To provide the most useful guidance on taper length and its relationship to speed differential when a turning vehicle clears the through lane, a study that explores lane-change location with respect to taper length would be very informative.
- Use of auxiliary/acceleration lanes for vehicles making a left-turn from a minor road. Some research has been conducted on this topic, but the conditions under which auxiliary/acceleration lanes provide the greatest safety and operational effects have not been fully explored. Guidance on when to provide acceleration lanes and how to determine the appropriate design dimensions has potential for widespread use in the design of rural high-speed intersections with two-way stop control.
- Design of channelization elements. The literature review and state practitioner interviews indicated that there is a need for more detailed guidance on the design of channelized right-turn islands and median openings. These designs have benefits for capacity and access, but they also have the potential to affect pedestrian crashes (at right-turn islands) and turning crashes (at median openings). A better understanding of what design dimensions provide the best operational and safety benefits will help designers know what elements are best suited for conditions at specific locations.
- Design differences between right-turn and left-turn lanes. It is commonly assumed that left-turn lanes and right-turn lanes are similar enough that most design elements can generally be designed the same way, but differences between the two types of lanes are not formally documented and it is not clear which differences are important enough to specifically include in the Green Book.
- Benefits of offset left-turn lanes on safety and other performance measures. Previous research has identified some operational benefits to offset left-turn lanes, along with suggestions for selected design elements. However, it is unclear what effect offset left-turn lanes have on safety, and whether there are other benefits (e.g., improvement in perceived sight distance, reduced driver workload).
- Appropriate distance between simultaneous left turns, especially for multiple turn lanes. Selected guidelines discuss the distance between opposing left-turn lanes, but a comprehensive study was not found in the review during this project. That distance is a particularly noteworthy design element in the design of intersections with simultaneous turns from double left-turn lanes.
- Auxiliary lanes in conjunction with passing lanes on two-lane highways. Prior studies have recommended avoiding locating high-traffic intersections and driveways within the boundaries of a passing lane, but no specific study on the use of auxiliary lanes within passing lane sections was found. Also, no study on tradeoffs between turning lanes and passing lanes was identified. Additional research would be beneficial in answering the question of when to include an auxiliary lane when a passing lane is present, or when to consider a left-turn or right-turn lane instead of a passing lane.
- Operational and safety benefits of storage length. As with some other treatments in this list of research topics, the *Highway Safety Manual* (*1*) notes that storage length is a treatment with unknown crash effects. The safety benefits of lengthening a left-turn lane are unclear, as are the conditions under which the queues from neighboring through lanes affect the design of auxiliary lane lengths at signalized intersections.
- Development of auxiliary lane pavement markings for use in multiple turn lane configurations. While this may be viewed as a pavement marking issue, proper location of these markings has the potential to have an impact on the operational characteristics of the multiple turning lanes configuration. Further, the 2009 *Manual on Uniform Traffic Control Devices* does not provide sufficient guidance for determining the beginning of turn lane lines within the transition area of multiple turn lanes. Without positive guidance for drivers within the approach transition, proper vehicle alignment is more difficult, causing more turbulent flow, and, thus, may influence operations.
- Unsignalized or permissive (e.g., green ball, flashing yellow arrow) phasing of left turns across more than two lanes has been a concern from a safety perspective due to:
  - Staggering/shadowing of opposing vehicles.
  - Opposing vehicle speed differentials.
  - Driver judgment.
  - Ability to clear opposing through traffic, especially for left turns across three or more lanes at speeds greater than 35 mph.

  Research could clarify the safety relationships between left turns and the number of opposing lanes for different geometric/traffic characteristics such as signalized or unsignalized, presence of left-turn lane, presence of permissive-only or permitted/permissive phasing when signalized, posted speed limit, and others.

- Entry angle for right-turn lane. The entry angle can affect how a right-turn lane operates and can affect safety on the approach. A flat-angle entry alignment requires a driver to look left over the shoulder to check for merging traffic and can result in the driver losing sight of a car still in front. The driver may accelerate to accept a gap, not realizing that the car in front is still there. Additional research and field evaluation should be conducted to determine the relationship between crashes and the angle of the right-turn lane to the cross street. The research should consider cross-street operating speed, cross-street volume, and the age of the driver.
- Impacts of trucks on turn lane design. The double left-turn lane study used saturation flow, which removed trucks from the analysis as recommended by the *Highway Capacity Manual.* The characteristics of trucks will obviously have an impact on saturation flow; however, the research need is how these truck characteristics affect safety and operations with respect to other design characteristics, such as lane width or median type. An example question is the following: Are the safety and operations of turn lanes more influenced by the available lane width when a larger percentage of trucks is present?

APPENDIX A

Recommended Revisions to AASHTO Green Book

Overview

This appendix presents the research team's suggested changes to the AASHTO *Policy on Geometric Design of Highways and Streets* (commonly known as the Green Book) (*2*). Proposed additions to existing content are shown with double underlines, and proposed deletions are shown with ~~strikethroughs~~. The proposed revisions are based on research conducted as part of this NCHRP project along with research recently completed.

Some of the proposed revisions include adding specific references to research reports or other sources; those sources can be found in the References list at the end of this document, just as the Green Book is referenced above. When revisions are formally added to the Green Book, the actual reference numbers of those sources should change to coincide with the existing numbering sequence in Chapter 9 of the Green Book. Proposed new or revised Green Book figures and tables are assigned a figure or table number within this appendix so that they will appear in the list of figures and list of tables at the front of this report. These titles also include the Green Book figure or title number that they would replace or they include an "X" number to indicate that they are new figures or tables.

9.3 Types and Examples of Intersections, Introductory Section (pages 9-8 to 9-10)

Proposed Revision to Green Book

The basic types of intersections are three-leg (T), four-leg, multi-leg, and roundabouts. Further classification of the basic intersection types includes such variations as unchannelized, flared, and channelized intersections as shown in Figure 9-3. Additional variations include offset intersections, which are two adjacent T-intersections that function similar to a four-leg intersection, and indirect intersections that provide one or more of the intersection movements at a location away from the primary intersection. At each particular location, the intersection type is determined primarily by the number of intersecting legs; the topography; the character of the intersecting highways; the traffic volumes, patterns, and speeds; and the desired type of operation. These characteristics are also related to the type of traffic control (e.g., traffic signal, two-way or all-way stop, yield on minor approach). Variations of these intersection types to improve capacity by providing indirect left-turn movements are addressed in Section 9.9 on "Indirect Left Turns and U-Turns."

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Although many of the intersection design examples are in urban areas, the principles involved apply equally to design in rural areas. Some minor design variations occur with different kinds of traffic control, but all of the intersection types shown lend themselves to cautionary or non-stop control, stop control for minor approaches, four-way stop control, and both fixed-time and traffic-actuated signal control. Right-turn roadways without stop or yield control are sometimes provided at channelized intersections. Such ~~free-flow~~ channelized right-turn lanes should be used only where an adequate merge is provided. Where motor vehicle conflicts with pedestrians or bicyclists are anticipated, provisions for pedestrians and bicycle movements must be considered in the design. Channelized right-turn lanes have a definite role in improving operations and safety at intersections; however, at locations with high pedestrian volumes ~~In built-up areas~~, the use of ~~free-flow~~ channelized right-turn lanes should be considered only where significant traffic capacity or safety problems may occur without them and adequate pedestrian crossings can be provided.

Discussion

The results from NCHRP Project 3-89 (Design Guidance for Channelized Right-Turn Lanes) (*28*) need to be integrated into this section of the Green Book. For example, NCHRP Project 3-89 recommended that the term "free-flow right turn" be replaced with the term "channelized right-turn lane." Another suggestion provided by others is to be more precise with the type of traffic control present, e.g., yield control, signal control, free flow (i.e., no traffic control).

9.3 Types and Examples of Intersections, 9.3.1 Three-Leg Intersections (pages 9-10 to 9-14)

Proposed Revision to Green Book

Channelized Three-Leg Intersections

Channelization is often desirable for some reasons described in Section 9.6.2. Where channelization is provided, islands and turning roadways should be designed to accommodate the wheel tracks of each vehicle movement while providing optimum crossing paths and storage for pedestrians within the proposed intersection. The simplest form of channelization is accomplished by increasing the corner radius between the two roadways sufficiently to permit a separate turning roadway that is separated from the normal traveled ways of the intersecting approaches by an island as shown in Figure 9-5A and 9-5C. The approach roadway may include a separate right-turn lane leading to the turning roadway for the accommodation of right-turn traffic. Often the provision for a separate lane for left turns or for through movements to bypass left-turning traffic is appropriate on two-lane highways where right-turning roadways are justified. Left-turning traffic can be accommodated by the flaring of the through highway as shown in Figure 9-5B and 9-5C. The right-turning roadways should be designed to discourage wrong-way entry while providing sufficient width for anticipated turning trucks.

Figure 9-5B depicts a channelized intersection incorporating one divisional island on the crossroad. Space for this island is made by flaring the pavement edges of the crossroad and by using larger-than-minimum pavement edge radii for right-turning movements. Figure 9-5C shows an intersection with a divisional island and right-turning roadways, a desirable configuration for intersections on important two-lane highways carrying intermediate to heavy traffic volumes (e.g., peak-hour volumes greater than 500 vehicles on the through highway with substantial turning movements). All movements through the intersection are accommodated on separate lanes.

Where the traffic demand at an intersection approaches or exceeds the capacity of a two-lane highway and where signal control may be needed in rural areas, it may be desirable to convert the two-lane highway to a divided section through the intersection, as shown in Figure 9-5C. In addition to adding auxiliary lanes on the through highway, the intersecting road (i.e., the stem of the three-leg intersection) may be widened on one or both sides for better maneuverability and increased capacity on the crossroad. The right-turn lane in the upper right quadrant accommodates a non-restricted exit from the major route.

Figures 9-4B and 9-5B provide examples of bypass lanes, which are added to the outside edge of the approach, allowing through vehicles to pass left-turning vehicles on the right, while Figures 9-4C and 9-5C show traditional left-turn lanes. Regardless of the treatment, consideration of traffic demand, delay savings, crash reduction, and construction costs are all key factors in determining whether to install a left-turn lane or a bypass lane. Research on left-turn accommodations at unsignalized intersections (9) produced warrants for the installation of left-turn lanes and bypass lanes that account for those factors.

Dimensions for turning roadways (e.g., lane width, taper length, deceleration and storage length) are provided in Section 9.7. Bypass lanes for through traffic should be designed with the same lane width as the width of the travel lane upstream and downstream of the intersection; the taper rate recommended in Section 9.7.2 for turning roadways can also be used for bypass lanes. Warrants for the installation of bypass lanes and left-turn lanes are provided in Section 9.7.3.

Discussion

The existing language on bypass lanes in this section does not provide information on installation warrants or design dimensions. While the warrants could be inserted here, it seems better suited for inclusion in Section 9.7.3. Therefore, the recommended additions to this section include reference to the recently completed research in NCHRP Report 745 (*9*) that developed the warrants and a cross-reference to Section 9.7.3 where the new information on warrants is proposed. In addition, a cross-reference to Section 9.7 is also provided to give guidance on appropriate design dimensions.

During the review of this section, researchers observed that revisions to some existing diagrams would also be beneficial. Specific suggestions for revision include:

- The left through lanes on the major road in Figure 9-3B appear to lead into the opposing left-turn lanes. The through lanes should be realigned.
- The lane line separating the eastbound through and left-turn lanes in Figure 9-4C appears to be missing, and the eastbound-to-northbound left-turn movement arrow in Figure 9-4B appears to turn into oncoming traffic. The figure should be revised accordingly.
- All of the figures in this section could benefit from inclusion of the conflict points in each intersection configuration.

A more detailed review and revision of these figures was not a point of emphasis in this review, but these suggested revisions are provided as information to be considered in a more formal review by AASHTO in the future.

9.3 Types and Examples of Intersections, 9.3.2 Four-Leg Intersections (pages 9-14 to 9-19)

Proposed Revision to Green Book

A flared intersection, illustrated in Figures 9-6B and 9-6C, has additional capacity for through and turning movements at the intersection. Auxiliary lanes on each side of the normal pavement at the intersection illustrated in Figure 9-6B enable through vehicles to pass slow-moving vehicles preparing to turn right. Depending on the relative volumes of traffic and the type of traffic control used, flaring of the intersecting roadways can be accomplished by parallel auxiliary lanes, as on the highway shown horizontally, or by pavement tapers, as shown on the crossroad. Flaring generally is similar on opposite legs. Parallel auxiliary lanes are essential where traffic volume on the major highway is near the uninterruptedflow capacity of the highway or where through and cross-traffic volumes are sufficiently high to warrant signal control. Auxiliary lanes are also desirable for lower volume conditions. The length of added pavement should be determined as it is for speed-change lanes, as shown in the subsection on Auxiliary Lanes in Section 9.7, and the length of uniform lane width, exclusive of taper, should normally be greater than 45 m [150 ft] on the approach side of the intersection. The length of the lane-addition and lane-drop tapers needed to accomplish the flaring can be determined from Equations 3-37 and 3-38 in Section 3.34.

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Typical configurations of four-leg intersections with simple channelization are shown in Figure 9-7. Right-turning roadways as shown in Figure 9-7A are often provided at major intersections for the more important turning movements, where large vehicles are to be accommodated, and at minor intersections in quadrants where the angle of turn ~~greatly exceeds~~ is substantially below 90 degrees as shown in Figure 9-8A.

A configuration with right-turn roadways in all four quadrants of the intersection as illustrated in Figure 9-7A is suitable where sufficient space is available and right-turn volumes are high. Where one or more of the right-turning movements need separate turning roadways, additional lanes are generally needed for the complementary left-turning movements.

The intersection with divisional islands on the crossroad illustrated in Figure 9-7B fits a wide range of volumes and its capacity is governed by the roadway widths provided through the intersection.

For an intersection on a two-lane highway operating near capacity or carrying moderate volumes at high speeds, a configuration with channelized left-turn lanes as shown in Figure 9-7C may be considered. The auxiliary lanes are used for speed changes, maneuvering, and storage of turning vehicles. The form of channelization on the crossroad should be determined based on the cross and turning volumes and the sizes of vehicles to be accommodated.

Where roadways cross one another at an angle other than 90 degrees, it is desirable to realign one or both highways to reduce the skew angle. Drivers may have difficulty seeing cross traffic at an intersection with a severe skew because of the added difficulty in turning their heads and the reduced visibility often created by parts of the vehicle. These effects are most pronounced for right-turn-on-red (RTOR) maneuvers at signalized intersections and for any maneuver from a minor road at two-way stop-controlled intersections.

Older drivers in particular have difficulty with skewed intersections, due to restricted range of motion and diminished reaction time. The *Highway Design Handbook for Older Drivers and Pedestrians (29)* recommends:

1. In the design of new facilities or redesign of existing facilities where right-of-way is not restricted, all intersecting roadways should meet at a 90-degree angle.
2. In the design of new facilities or redesign of existing facilities where right-of-way is restricted, intersecting roadways should meet at an angle of not less than 75 degrees.
3. At skewed intersections where the approach leg to the left intersects the driver's approach leg at an angle of less than 75 degrees, the prohibition of RTOR is recommended.

Figure 9-8A shows use of right-turn islands and roadways at an intersection in quadrants where the angle of intersection is substantially below 90 degrees. Figure 9-8B shows an oblique intersection that has been modified to reduce the skew with separate turning roadways in the acute angle quadrants. When realignment cannot be obtained, extensive application of appropriate signing and signal control is recommended.

The simplest form of intersection on a divided highway has paved areas for right turns and a median opening conforming to designs discussed throughout this chapter. Sections 9.4 through 9.11 include guidelines to be used for intersection design. Often the speeds and volumes of through and turning traffic justify a higher type of channelization suitable for the predominant traffic movements. Channelization is often used at intersections on divided highways as shown in Figure 9-9.

Right-turning roadways with speed-change lanes and median lanes for left turns afford both a high degree of efficiency in operation and high capacity and permit through traffic on the highway to operate at reasonable speed.

Figure 9-9B shows an intersection configuration with dual left-turn lanes for each of the left-turning movements. This configuration needs traffic signal control with a separate signal phase for the dual left-turn movement. Dual left-turn lanes may be used for any one approach or a combination of approaches for which the left-turn volumes are high. The auxiliary lanes in the median may be separated from the through lanes by pavement markings or by an elongated island, as shown for the east-west direction in Figure 9-9B. Furthermore, pavement markings, contrasting pavements, and signs should be used to discourage through drivers from entering the median lane inadvertently. Left-turning vehicles typically leave the through lane to enter the median lane in single file but, once within it, are stored in two lanes. On receiving the green signal indication, left-turn maneuvers are accomplished simultaneously from both lanes. The median opening and the crossroad pavement should be sufficiently wide to receive the two side-by-side traffic streams.

~~Where roadways cross one another at an angle other than 90 degrees, the effects of the skew can be mitigated by providing right-turn roadways or realigning the cross street to reduce the impact of the skew. Figure 9-8A shows use of right-turn islands and roadways at an intersection in quadrants where the angle of intersection greatly exceeds 90 degrees. Drivers have difficulty seeing cross traffic at an intersection with a severe skew because of the difficulty drivers, particularly older drivers, have in turning their heads and the reduced visibility often created by parts of the vehicle. It is desirable to realign one or both highways to reduce the skew angle. Figure 9-8B shows an oblique intersection that has been modified to reduce the skew with separate turning roadways in the acute angle quadrants. When realignment cannot be obtained, extensive application of appropriate signing and signal control is recommended.~~

Discussion

The text mentions briefly that there is a suggested length for auxiliary lanes at flared intersections but does not really specify what that is or how to determine it, except to say, "The length of added pavement should be determined as it is for speed-change lanes and the length of uniform lane width, exclusive of taper, should normally be greater than 45 m [150 ft] on the approach side of the intersection." Speed-change lanes, it is assumed, refer to auxiliary lanes at interchanges, but no cross-reference is made; one could assume that instead of referring to intersection auxiliary lanes in Section 9.7, it refers to Tables 10-3 through 10-5 and/or the discussion on interchange auxiliary lanes on pp. 10-76. The guidance given to the reader could be much improved, to describe not only the needed lengths of the full-width lane upstream and downstream of the intersection but also the appropriate lane-addition and lane-drop tapers, which could be drawn from the L = WS equations (3-37 and 3-38) on pp. 3-134.

The discussion of the effects of skew and how it can be mitigated could lead to confusion or incorrect conclusions (compare with assessment of Section 9.4.2). In addition, the supporting text on skew should be moved closer to Figure 9-8, where the examples are given. Finally, the text incorrectly describes turning roadways being provided where the turning angle is greater than 90 degrees. Rewriting the text and redrawing the figures will resolve the discrepancies.

The suggested revisions correct the error in the text about turning roadways on quadrants greater than 90 degrees, revise the discussion on skew to be consistent with the *Highway Design Handbook for Older Drivers and Pedestrians* (27), and revise text in Section 9.4.2 to have the information on skew closer to the figure that illustrates it. In addition to the *Highway Design Handbook for Older Drivers and Pedestrians*, see references under Section 9.4.2.

9.4 Alignment and Profile, 9.4.2 Alignment (page 9-27)

Proposed Revision to Green Book

> Once a decision has been made to realign a minor road that intersects a major road at an acute angle, the angle of the realigned intersection should be as close to 90 degrees as practical. Although a right-angle crossing is normally desired, some deviation from a 90-degree angle is permissible. Reconstruction of an intersection to provide an angle of at least ~~60~~ 75 degrees provides most of the benefits of a 90-degree intersection angle while reducing the right-of-way takings and construction costs often associated with providing a right-angle intersection.

Discussion

There is no citation in the existing text to back up the claim that "an angle of at least 60 degrees provides most of the benefits of a 90-degree intersection angle," and review of existing literature did not uncover any substantiating research. However, recent research into both older driver characteristics and driver field of view considerations contradicts this statement and, instead, recommends either 75 or 70 degrees. While some of the studies are older, their results are substantiated by newer research and, thus, were included in the list below. Further, many states' design manuals recommend either 75 or 70 degrees, and a sampling of these references are included below. Finally, another section of Chapter 9 specifically calls for an intersection angle of not less than 75 degrees. In Section 9.6.5, Turning Roadways with Corner Islands, Subsection Oblique-Angle Turns with Corner Islands, on page 9-112, the Green Book states, "If practical, angles of intersection less than 75 degrees should not be used." References that support the use of recommending a limit of 70 or 75 degrees include the following:

- FHWA *Highway Design Handbook for Older Drivers and Pedestrians* (*29*), in I. Intersections (At-Grade), states, "In the design of new facilities or redesign of existing facilities where right-of-way is restricted, intersecting roadways should meet at an angle of not less than 75 degrees."
- ITE, *Traffic Engineering Handbook* (5th edition) (*92*) states on page 385, "Crossing roadways should intersect at 90 degrees if possible, and not less than 75 degrees." The handbook also says, "Skew angles in excess of 75 degrees often create special problems at stop-controlled rural intersections. The angle complicates the vision triangle for the stopped vehicle; increases the time to cross the through road; and results in a larger, more potentially confusing intersection."
- Gattis and Low (*106*) suggested from a driver field of view study that 70 degrees is more appropriate because with greater skew angles portions of the vehicle block driver line of sight.
- Son et al. (*45*) developed method to calculate sight distance available to drivers at skewed intersections and directly considered intersection angle, lane width, shoulder width, position of stop line, vehicle dimension, and driver's field of view. Results include tables listing sight distance available based on skew and support skew angle of 70 degrees.
- Garcia and Libreros (*107*) and Garcia (*108*) conducted research based on driver's field of view and recommended skew angles of no less than 70 degrees for crossing maneuvers.
- Ohio DOT, *Location and Design Manual Volume 1—Roadway Design* (*109*), Section 401.3 Crossroad Alignment, states, "Intersection angles of 70 degrees to 90 degrees are to be provided on all new or relocated highways. An angle of 60 degrees may be satisfactory if: (1) the intersection is signalized; or (2) the intersection is skewed such that a driver stopped on the side road has the acute angle (at center of intersection) on his left side (vision not blocked by his own vehicle)."
- Wisconsin DOT, *Facilities Development Manual* (*110*), in Chapter 11 Design, Section 25 Intersections at Grade, Subsection 2.8.1 (FDM 11-25-2.8.1), says that for intersections on a tangent or on outside of curve the desirable skew angle

is between 75 degrees and 105 degrees with a minimum of 70 degrees and maximum of 110 degrees.

- Illinois DOT, *Bureau of Local Roads and Streets Manual* (*111*) in Chapter 34, Intersections, states, "Preferably, the angle of intersection should be within 15° of the perpendicular. This amount of skew can often be tolerated because the impact on sight lines and turning movements is not significant. Under restricted conditions where obtaining the right-of-way to straighten the angle of intersection would be impractical, an intersection angle up to 30° from the perpendicular may be used."
- California DOT, *Highway Design Manual* (*112*), in Section 403.3 Angle of Intersection, states, "When a right angle cannot be provided due to physical constraints, the interior angle should be designed as close to 90 degrees as is practical, but should not be less than 75 degrees."
- Harkey (*47*) recommended that the minimum critical angle for intersections in roadway design policies be revised to 75 degrees.

9.4 Alignment and Profile, 9.4.2 Profile (page 9-27)

Proposed Revision to Green Book

The calculated stopping and accelerating distances for passenger cars on grades of 3% or less differ little from the corresponding distances on the level. Grades steeper than 3% may need changes in several design elements to sustain operations equivalent to those on level roads. Most drivers are unable to judge the effect of steep grades on stopping or accelerating distances. Their normal deductions and reactions may thus be in error at a critical time. Accordingly, grades in excess of 3% should be avoided on the intersecting roads in the vicinity of the intersection. Where conditions make such designs too expensive, grades should not exceed about 6 percent, with a corresponding adjustment in specific geometric design elements.

The United States Access Board provides proposed minimum design standards that are to be applied to all public rights-of-way, including sidewalks and crosswalks (*113*) and shared-use paths (*57*). These guidelines specify that the cross-slope of a sidewalk should not exceed 2% measured perpendicular to the direction of pedestrian travel. This may necessitate tabling of the intersection area, which will impact the vertical alignment of the roadway and may affect intersection drainage.

Discussion

What design elements need changed? How does one quantify "vicinity"? Which "specific geometric elements" apply? There is no citation given for this section to reference the supporting documentation for these terms. As such, a research project that generates a list of design elements, a quantification of vicinity, and a list of specific elements should be developed to support these statements, or they should be dropped or modified.

Further, the Americans with Disabilities Act (ADA) requires that public rights-of-way, including sidewalks and crosswalks, be accessible to pedestrians with disabilities. ADA guidelines require that the cross-slope of a sidewalk should not exceed 2% measured perpendicular to the direction of pedestrian travel. This necessitates tabling of an intersection that impacts the vertical alignment of the roadway and can impact drainage. Text was added to cover this point. The ADA guidelines are evolving with the most current versions available at the following links:

- Published in the Federal Register on July 26, 2011: http://www.access-board.gov/guidelines-and-standards/streets-sidewalks/public-rights-of-way/proposed-rights-of-way-guidelines
- Published in the Federal Register on February 13, 2013: http://www.access-board.gov/guidelines-and-standards/streets-sidewalks/shared-use-paths/supplemental-notice

9.6 Turning Roadways and Channelization, 9.6.1 Types of Turning Roadways (pages 9-55 to 9-56)

Proposed Revision to Green Book

General

The widths of turning roadways for intersections are governed by the volumes of turning traffic and the types of vehicles to be accommodated. In almost all cases, turning roadways are designed for use by right-turning traffic. The widths for right-turning roadways may also be applied to other roadways within an intersection. There are three typical types of right-turning roadways at intersections: (1) a minimum edge-of-traveled-way design, (2) a design with a corner triangular island, and (3) a free-flow design using a simple radius or compound radii. The turning radii and the pavement cross slopes for free-flow right turns are functions of design speed and type of vehicles. For an in-depth discussion of the appropriate design criteria, see Chapter 3.

Channelized Right-Turn Lanes

Channelized right-turn lanes have a definite role in improving operations and safety at intersections. However, to achieve these benefits, they should have consistent design and traffic control and should be used at appropriate locations.

Crosswalk location—A pedestrian crosswalk could potentially be placed at any location along a channelized right-turn roadway (e.g., upstream, center, or downstream). It is obviously desirable to place the crosswalk at whatever location would maximize safety, presumably the location where pedestrians who are crossing or about to cross the right-turn roadway are most visible to motorists and where motorists are most likely to yield to pedestrians. An evaluation of crosswalks at existing channelized right turns (26) revealed that the majority of the sites (nearly 70 percent) had marked crosswalks near the center of the channelized right-turn lane; only about 30% of crosswalks were at the upstream or downstream end of the channelized right-turn lane. Similarly, a highway agency survey (11) found that highway agencies prefer a crosswalk location near the center of a channelized right-turn lane; over 70% of highway agencies reported in the survey that their practice was to place crosswalks near the center of channelized right-turn lanes.

Consistency of crosswalk location at channelized right-turn lanes is important to pedestrians with vision impairment, and current highway agency practice indicates a preference for crosswalk locations near the center of a channelized right-turn lane. A crosswalk location at the center of the channelized right-turn lane moves vehicle-pedestrian conflicts away from both the diverge maneuver at the upstream end of the channelized right-turn lane and the merge maneuver at the downstream end of the channelized right-turn lane. The only potential exception to a center crosswalk location for channelized right-turn lanes is where a Stop sign or traffic signal control is provided at the entry to the cross street; the crosswalk should be beyond the stop line at that point. To summarize the recommended guidance for the placement of crosswalks at channelized right-turn lanes:

- Where the entry to the cross street at the downstream end of the channelized right-turn lane has yield control or no control, place the crosswalk near the center of the channelized right-turn lane.
- Where the entry to the cross street at the downstream end of the channelized right-turn lane has Stop sign control or traffic signal control, place the crosswalk immediately downstream of the stop bar, where possible. Where the channelized right-turn roadway intersects with the cross street at nearly a right angle, the stop bar and crosswalk can be placed at the downstream end of the channelized right-turn roadway.

Special crosswalk signing and marking—Marked crosswalks are the primary means of indicating the presence of a pedestrian crossing. However, drivers do not always yield the right of way to pedestrians simply because they are in a crosswalk. Other special crosswalk signing and marking treatments have been considered for use at pedestrian crossings on channelized right-turn roadways to enhance crossing safety for pedestrians, in general, and for pedestrians with vision impairment. These include:

- Use of a crosswalk to improve the visibility of the crosswalk for motorists and to better define crosswalk boundaries for pedestrians (raised crosswalks are particularly helpful to pedestrians with vision impairment).
- Addition of fluorescent yellow-green signs both at the crosswalk and in advance of the crossing location (to supplement the high-visibility markings).
- Use of a real-time warning device to indicate to the motorist when a pedestrian is present in the area (may be activated via passive detection technologies such as microwave or infrared or via traditional methods such as push buttons).
- Use of dynamic message signs (for real-time or static warning messages to motorists).

Additional signing and pedestrian crosswalk treatments may improve the motorist yield behavior and pedestrian use of the crosswalk.

Island type—A channelized right-turn lane consists of a right-turning roadway at an intersection, separated from the through travel lanes of both adjoining legs of the intersection by a channelizing island. At right-angle intersections, such channelizing islands are roughly triangular in shape, although the sides of the island may be curved, where appropriate, to match the alignment of the adjacent roadways. Islands serve three primary functions: (a) channelization—to control and direct traffic movement, usually turning; (b) division—to divide opposing or same-direction traffic streams; and (c) refuge—to provide refuge for pedestrians. Most islands combine two or all of these functions. Islands for channelized right-turn lanes typically serve all three functions.

The edges of channelizing islands may be defined by raised curbs or may consist of painted pavement or turf that is flush with the pavement. Most channelizing islands in urban areas are defined by raised curbs. Curbed islands are considered most favorable for pedestrians because curbs most clearly define the boundary between the traveled way, intended for vehicle use, and the island, intended for pedestrian refuge. Curbed islands can improve the safety for pedestrians by allowing them to cross the street in two stages. Raised islands with cut-through pedestrian paths are important to pedestrians with vision impairment because they provide better guidance and information about the location of the island than painted islands. Where curb ramps are provided, truncated dome detectable warnings are required at the base of the ramp, where it joins the street, to indicate the location of the edge of the street to pedestrians with vision impairment.

Radius of turning roadway—Design criteria for the radii of channelized right-turn roadways are a function of turning speeds, truck considerations, pedestrian crossing distances, and resulting island sizes. Channelized right-turn lanes provide one method for accommodating larger turning radii without widening the major-street pedestrian crossings and without increasing the intersection pavement area. Where right-turn volumes are high and pedestrian and bicycle volumes are relatively low, capacity considerations may dictate the use of larger radii, which enable higher-speed, higher-volume turns. However, small turning radii, which promote low-speed right turns, are appropriate where such turns regularly conflict with pedestrians, as higher speeds have been shown to result in a decrease in yielding to pedestrians by motorists.

Angle of intersection with cross street—The alignment of a channelized right-turn lane and the angle between the channelized right-turn roadway and the cross street can be designed in two different ways:

- A flat-angle entry to the cross street.
- A nearly right-angle entry to the cross street.

The two designs differ in the shape of the island that creates the channelized right-turn lane. The flat-angle entry design has an island that is typically shaped like an equilateral triangle (often with one curved side), while the nearly right-angle design is typically shaped like an isosceles triangle. The flat-angle entry design is appropriate for use in channelized right-turn lanes with either yield control or no control for vehicles at the entry to the cross street. The nearly right-angle entry design can be used with Stop sign control or traffic signal control for vehicles at the entry to the cross street; yield control can also be used with this design where the angle of entry and sight distance along the cross street are appropriate.

Deceleration lanes—Drivers making a right-turn maneuver at an intersection are usually required to reduce speed before turning. Significant deceleration that takes place directly on the through traveled way may disrupt the flow of through traffic and increase the potential for conflicts with through vehicles. To minimize deceleration in the through travel lanes, deceleration lanes should be considered. Right-turn deceleration lanes provide one or more of the following functions (*5*):

- A means of safe deceleration outside the high-speed through lanes for right-turning traffic.
- A storage area for right-turning vehicles to assist in optimization of traffic signal phasing.
- A means of separating right-turning vehicles from other traffic at stop-controlled intersection approaches.

The addition of a deceleration lane at the approach to a channelized right-turn lane provides an opportunity for motorists to safely slow down prior to reaching the crosswalk area at the turning roadway.

Acceleration lanes—Acceleration lanes provide an opportunity for vehicles to complete the right-turn maneuver unimpeded and then accelerate parallel to the cross-street traffic prior to merging. Channelized right-turn lanes with acceleration lanes appear to be very difficult for pedestrians with vision impairment to cross. Therefore, the use of acceleration lanes at the downstream end of a channelized right-turn lane should generally be reserved for locations where no pedestrians or very few pedestrians are present. Typically, these would be locations without sidewalks or pedestrian crossings; at such locations, the reduction in vehicle delay resulting from addition of an acceleration lane becomes very desirable.

Pedestrian signals—Pedestrian signals can be used at pedestrian crossings on channelized right-turn roadways to enhance crossing safety for pedestrians, particularly for pedestrians with vision impairment. Where a signal is provided for pedestrians to cross a channelized right-turn lane, a pedestrian-actuated signal should be considered.

Minimum Edge-of-Traveled-Way Designs

Where it is appropriate to provide for turning vehicles within minimum space, as at unchannelized intersections, the corner radii should be based on minimum turning path of the selected design vehicles. The sharpest turn that can be made by each design vehicle is shown in Sections 2.1.1 and 2.1.2, and the paths of the inner rear wheel and the front overhang are illustrated. The swept path widths indicated in Section 2.1.2, which are slightly greater than the minimum paths of nearly all vehicles in the class represented by each design vehicle, are the minimum paths attainable at speeds equal to or less than 15 km/h [10 mph] and consequently offer some leeway in driver behavior. These turning paths of the design vehicles shown in Figures 2-1 through 2-9 and Figures 2-13 through 2-23 are considered satisfactory as minimum designs. Tables 9-15 and 9-16 summarize minimum-edge-of-traveled-way design values for various design vehicles.

Discussion

NCHRP 3-89 (*28*) recommends inserting a new subsection on channelized right-turn lanes, given that the proposed level of detail is currently not found in the Green Book. The proposed subsection would be inserted between the existing subsections entitled "General" and "Minimum Edge-of-Traveled-Way Designs."

9.6 Turning Roadways and Channelization, 9.6.1 Types of Turning Roadways (page 9-80)

Proposed Revision to Green Book

From the analysis of these maneuvers and corresponding paths, together with other pertinent data, the appropriate type of minimum design can be selected. Applications of minimum designs for turning movements are common, even in rural areas. Minimum designs are appropriate for locations with low turning speeds, low turning volumes, ~~or~~ high property values, or environmental issues.

Discussion

NCHRP Synthesis 422 (*101*) showed that states primarily use three factors for justifying tradeoffs in highway geometric design: safety, cost, and environmental issues. As such, this consideration should also be added to the provided list of factors.

9.6 Turning Roadways and Channelization, 9.6.1 Types of Turning Roadways (page 9-88)

Proposed Revision to Green Book

~~The dimensions presented in Figures 9-34 and 9-35 demonstrate why curb radii of only 3 to 4.5 m [10 to 15 ft] have been used in most cities.~~ Curb radii should accommodate the expected amount and type of traffic and allow for safe turning speeds at intersections. Figures 9-34 and 9-35 show the impact on the right-turning path for typical curb radii designs used in many cities. A curb radius of 4.5 m [15 ft] is typically used for the intersection of a residential street with another residential street, collector, or arterial, while a curb radius of 7.5 m [25 ft] is typically used for the intersection of arterial streets or at locations that are truck or bus routes. Where larger radii are used, an intermediate refuge or median island is desirable or crosswalks may need to be offset so that crosswalk distances ~~are not objectionable~~ do not adversely impact intersection operation and safety. Typically, refuge islands are provided when the crossing distance exceeds 18.2 m [60 ft]. In summary, the corner radii proposed at an intersection on urban arterial streets should satisfy the needs of the drivers using them, the amount of right-of-way available, the angle of turn between the intersection legs, the number of pedestrians using the crosswalk, the width and number of lanes on the intersecting street, and the posted speeds on each street. The following is offered as a guide:

Discussion

It is not clear how the figures demonstrate the reason for selected radii. What they do show is the right-turning paths of various design vehicles superimposed upon typical curb radii found in many U.S. cities. This typical curb radius is 15 ft for residential street to residential/arterial street and 25 ft for intersections of arterial streets or locations that are truck/bus routes.

The section cautions designers to ensure that crosswalk distances are not "objectionable" but does not quantify how this is determined. Sources for supporting information include the following:

- AASHTO, *Guide for the Planning, Design, and Operation of Pedestrian Facilities* (*114*).
- "A pedestrian pushbutton should be placed in the median of signalized mid-block crossings where the crossing distance exceeds 60 feet (18.2 meters)." Virginia DOT *Guidelines for the Installation of Marked Crosswalks* (*115*) on page 16.
- "It has been proposed that pedestrian refuge islands be provided wherever possible, when the total length of a crosswalk is greater than 75 feet, or in areas where there are many elderly or handicapped pedestrians." ITE, *Design and Safety of Pedestrian Facilities* (*116*) on page 14.

9.6 Turning Roadways and Channelization, 9.6.2 Channelization (page 9-93)

Proposed Revision to Green Book

- Motorists should not be confronted with more than one decision at a time; as such, sufficient median storage should be provided to permit through and left-turning traffic to make a two-stage maneuver.

Discussion

Crossing a divided highway or turning left from the crossroad onto a divided highway involves more than one decision if there is not sufficient median storage to break the movement into two phases. This maneuver is further complicated by the larger scan area created by a divided highway.

9.6 Turning Roadways and Channelization, 9.6.2 Channelization (page 9-93)

Proposed Revision to Green Book

- Where the distance to the downstream driveway or intersection is less than the desirable distance for merging or weaving and where pedestrians are present, turning roadways should be controlled with a yield, stop, or signal control and the angle of the intersection should be greater than ~~60~~ 75 degrees.
- Traffic streams that intersect without merging and weaving should intersect at angles as close to 90 degrees as practical, with a range of ~~60~~ 75 to ~~120~~ 105 degrees acceptable.

Discussion

Intersection skew angle should be changed to 75 degrees. Please refer to detailed discussion above in Section 9.4 for details and supporting documentation.

9.6 Turning Roadways and Channelization, 9.6.2 Channelization (page 9-94)

Proposed Revision to Green Book

- ~~Refuge~~ areas for turning vehicles should be provided separate from through traffic.
- For locations with sufficient turning volumes and/or safety concerns, separate storage lanes should be used to permit turning traffic to wait clear of through-traffic lanes.

Discussion

The use of "refuge" to describe the storage area for the turning vehicles is confusing, as it is typically used in reference to areas to protect pedestrians or bicyclists. The use of separate storage lanes instead helps reduce that confusion and is more descriptive. Further, there needs to be sufficient traffic present to warrant the use of a separate turn lane before one should be required.

9.6 Turning Roadways and Channelization, 9.6.3 Islands (page 9-97)

Proposed Revision to Green Book

Widening a roadway to include a divisional island (Figure 9-37) should be done so that the proper paths to follow are unmistakably evident to drivers. The alignment should require no appreciable conscious effort in vehicle steering. Often the highway is on a tangent, and to introduce dividing islands, reverse-curve alignment would be needed. Tapers can be used, but should be consistent with lane shifts at the design speed. In rural areas, where speeds are generally high, reversals in curvature should preferably be with radii of 1165 m [3,825 ft] or greater. Sharper curves may be used on intermediate-speed roads (up to 70 km/h [45 mph]) with radii of 620 m [2,035 ft] or greater. Usually, the roadway in each direction of travel is bowed out, more or less symmetrically about the centerline as shown in Figure 9-37A. Widening may also be implemented on one side only with one of the roadways continuing through the intersection on a straight course (see Figure 9-37 B). When this arrangement is used for a two-lane road planned for future conversion to a divided highway, the traveled way on tangent alignment will become a permanent part of the ultimate development.

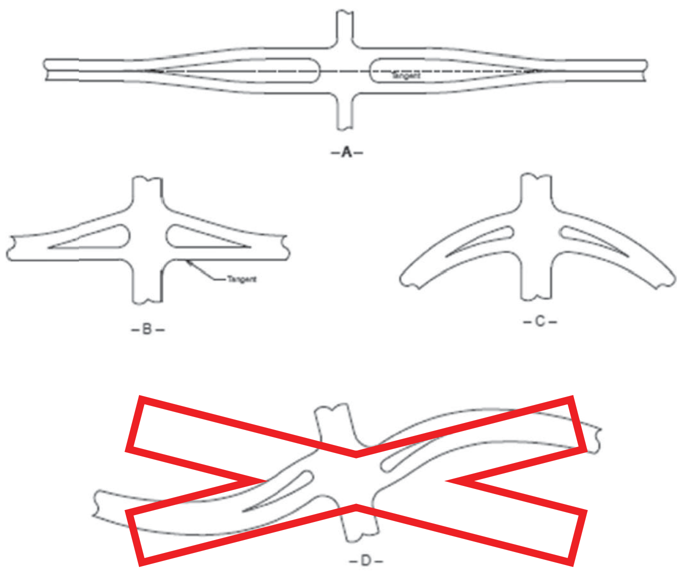


Figure 9-37. Alignment for Addition of Divisional Islands at Intersections

Source: *A Policy on Geometric Design of Highways and Streets* (2011) by the American Association of State Highway and Transportation Officials, Washington, D.C. Used by permission.

Figure A-1. Green Book Figure 9-37 with suggested deletion of image D.

Widening on tangent alignment, even with flat curves, may produce some appearance of distorted alignment. Where the road is on a curve or on widening alignment, advantage should be taken of the curvature in spreading the traffic lanes without using reverse curves, as illustrated in ~~sections C and D of~~ Figure 9-37C.

Discussion

Figure 9-37 (reproduced as Figure A-1) needs to be improved. Currently, B makes it appear that the taper used is insufficient, while D is drawn in such a way as to make it appear that the alignment for approaching traffic is directed into the opposite direction through lanes at the intersection, which would promote wrong-way movements. As such, B (and perhaps C, which also shows poor alignment) should be replaced with better representations of the concepts being illustrated. The word "tangent" in A should be changed to "centerline" to match the word used in the text.

9.6 Turning Roadways and Channelization, 9.6.3 Islands (page 9-104)

Proposed Revision to Green Book

Delineation is especially pertinent at the approach nose of a divisional island. In rural areas, the approach should consist of a gradual widening of the divisional island as indicated in Figure 9-41. Although not as frequently obtainable, this same design also should be striven for in urban areas. Preferably, the approach should gradually change to a raised surface with texture or to jiggle bars that may be crossed readily even at considerable speed. The transition taper length should be computed with Equation 3-37 {page 3-134 in 2011 Green Book} where the posted or statutory speed limit is 70 km/h [45 mph] or greater and with Equation 3-38 (pp. 3-134 in 2011 Green Book) where the posted or statutory speed limit is less than 70 km/h [45 mph]. If this distance cannot be met, this transition section should be as long as practical. The cross sections in Figure 9-41 demonstrate the transition. The face of curb at the approach island nose should be offset at least 0.5 m [2 ft] and preferably 1 m [3 ft] from the normal edge of traveled way, and the widened pavement gradually should be transitioned to the normal width toward the crossroad.

Discussion

There is no specific guidance given regarding the length of transition except to say that it "should be as long as practical." Most state design manuals use a form of the taper equations contained on page 3-134 as Equation 3-37 and Equation 3-38 as shown in the following sources:

- "An approach taper directs traffic to the right. Approach taper lengths are calculated using the following: Design Speed of 50 mph [80 km/h] or more: L = WS [L = 0.6 WS]. Design Speed less than 50 mph [80 km/h]: L = WS^2/60, (L = WS^2/156). Where: L = Approach taper length in feet [m] W = Offset width in feet [m] S = Design Speed [km/h]." Ohio DOT, *Location and Design Manual Volume 1—Roadway Design* (*109*) in Section 401.6 Approach Lanes.
- "The length of the Approach Taper varies depending on the operating speeds. Guidelines for determining lengths are: For speeds of 45 mph (70 kph) and over: L = WS, (L = 0.6 WS). For speeds under 45 mph (70 kph): L = WS^2/60, (L = WS^2/100). Where: L = Length of entering taper, ft (m) W = Width to be tapered, ft (m) S = Operating Speed, mph (kph)." South Dakota DOT, *Road Design Manual* (*117*) in Chapter 12 Intersection, on page 12-22.
- In Iowa DOT *Design Manual*, Chapter 6: Geometric Design: "The procedure for determining minimum taper ratios for redirecting through lanes is the same as shown in Table 1 for lane drops; however, for design speeds over 45 mph (70 km/h), the use of reverse curves rather than tapers is recommended." Text support for Table 1 [reproduced as Table A-1 below] following: "When dropping a through lane, the minimum length of taper can be determined by the following formulas: L = WS^2/60 for speeds of 40 mph or less (L = WS^2/155 for speeds of 70 km/h or less) L = S × W for speeds of 40 mph or more (L = 0.62 × S × W for speeds of 70 km/h or more) where L = minimum length of taper, S = posted speed limit or 85th percentile speed, W = width of lane to be dropped or redirection offset. Preferably, taper

Table A-1. Length and taper ratio for dropping 12-ft lane (*118*).

English units

Design Speed (mph)	30	35	40	45	50	55	60	65	70
Taper Ratio	15:1	25:1	30:1	45:1	50:1	55:1	60:1	65:1	70:1
Length (L) in feet	180	300	360	540	600	660	720	780	840

metric units

Design Speed (km/h)	45	55	65	70	80	90	100	110	120
Taper Ratio	15:1	20:1	30:1	45:1	50:1	60:1	65:1	70:1	75:1
Length (L) in meters	54	72	108	162	180	216	234	252	270

ratios should be evenly divisible by 5 (15:1, 20:1, etc.). Calculations that result in odd ratios should be rounded up to the next increment of 5" (*118*). "Table 1 utilizes the formulas to determine the appropriate taper ratios for dropping a 12-foot (3.6 meter) wide lane. The ratio remains constant for a given design speed, while the length varies with the lane width" (*118*).

- Georgia DOT, in Section 4.2.4 Lane Width Transitions and Shifts, states, "Lane width transitions can occur at several locations including: . . . Mainline lane shifts in advance of a typical section change such as a change in median width. There are two methods by which an alignment transition or "shift" may be accomplished: The first method is to treat the transition or shift as though it were any other alignment change. With this approach, a transition or shift would be accomplished through the use of a series of reverse curves. Quite often, the use of curve radii which do not require superelevation result in a length of transition greater than that required by providing a taper. Superelevation should be used if warranted by normal procedures. The second method of accomplishing a transition or "shift" involves the use of tapers. Tapers are acceptable provided the following two conditions exist: The alignment shift is consistent with the cross-slope of the roadway and does not require "shifting" over the top of an existing pavement crown. The direction of the shift is not counter to the pavement cross-slope (from a superelevation or reverse-crown consideration). Taper lengths associated with shifts on Georgia roadways should be calculated as: Case 1—Design Speed $\geq$ 45 mph: $L = W \times S$; Case 2—Design Speed < 45 mph: $L = (W \times S^2)/60$ Where L = distance needed to develop widening (ft), W = width of lane shift (ft), s = design speed (mph)" (*119*).
- Washington State DOT *Design Manual* in Chapter 1310 Intersections, on page 1310-20 in Exhibit 1310-10a Median Channelization: Widening (*120*), provides a table of desirable taper rates, reproduced as Table A-2.
- While the formulas are not directly presented as part of the approach taper length discussion, the taper rates shown in the table are, for the most part, directly attributable to the formulaic methodology.

Table A-2. Desirable taper rates for widening for median channelization (*120*).

Posted Speed	Desirable Taper Rate [6]
55 mph	55:1
50 mph	50:1
45 mph	45:1
40 mph	40:1
35 mph	35:1
30 mph	30:1
25 mph	25:1

9.6 Turning Roadways and Channelization, 9.6.4 Free-Flow Turning Roadways at Intersections (page 9-106)

Proposed Revision to Green Book

~~An important part of the design on some intersection is the design of a free-flow alignment for right turns. Ease and smoothness of operation can result when the free-flow turning roadway is designed with compound curves preceded by a right-turn deceleration lane, as indicated in Figures 9-42B and 9-42C. The shape and length of these curves should be such that they: (1) allow drivers to avoid abrupt deceleration, (2) permit development of some superelevation in advance of the maximum curvature, and (3) enable vehicles to follow natural turning paths. The design speed of a free-flow turning roadway for right turns may vary between the end of the right-turn deceleration lane and the central section. The design speed of the turning roadway may be equal to, or possibly within 20 to 30 km/h [10 to 20 mph] less than the through roadway design speed. Refer to Tables 3-8 through 3-12 for minimum radii for right-turning traffic. Turning roadways at intersections should use the "upper range" design speed whenever practical although the "middle range" speeds may be used in constrained situations.~~

~~Figure 9-42 Use of Simple and Compound Curves at Free-Flow Turning Roadways~~

Discussion

This section and Figure 9-42 are out of place here. This information deals more with the exit design from either a freeway or grade-separated arterial and is already covered in Chapter 10 (Section 10.9.6 Ramps). Further, it confuses the issue with relation to corner islands. As such, it should be removed.

9.7 Auxiliary Lanes, 9.7.1 General Design Considerations (pages 9-124 to 9-125)

Proposed Revision to Green Book

Deceleration lanes are advantageous on higher-speed roads, because the driver of a vehicle leaving the highway has no choice but to slow down on the through-traffic lane if a deceleration lane is not provided. The failure to brake by the following drivers, because of a lack of alertness, may result in rear-end collisions.

~~Acceleration lanes are not always desirable at stop-controlled intersections where entering drivers can wait for an opportunity to merge without disrupting through traffic. Acceleration lanes are advantageous on roads without stop control and on all high-volume roads even with stop control where openings between vehicles in the peak-hour traffic streams are infrequent and short~~, particularly on roads with higher operating speeds and/or higher volumes.

Acceleration lanes are not desirable at all-way stop-controlled intersections where entering drivers can wait for an opportunity to merge without disrupting through traffic. For additional design guidance related to lengths and other aspects of deceleration and acceleration auxiliary lanes, refer to Section 10.9.6.

Discussion

The description of acceleration lanes at stop-controlled intersections is confusing; it looks like the word "always" should have been "all-way" to describe the traffic control. The following sentence about using acceleration lanes on stop-controlled approaches in peak periods seems to be a mixture of two different thoughts; if a roadway is stop-controlled, it does not need an acceleration lane because the Stop sign creates gaps, but if a roadway is not stop-controlled, then the high volumes and/or speeds in a peak period would be good cause for an acceleration lane for traffic entering from the minor road.

The existing two-sentence discussion on acceleration lanes appears to be merely an introduction leading into the cross-reference for Section 10.9.6. It is recommended to separate it into its own paragraph to help distinguish it from the discussion on deceleration lanes. Interestingly, deceleration gets its own section immediately following in Section 9.7.2, while the remainder of discussion on acceleration lanes is pretty much confined to Chapter 10, where it is described in the context of interchange ramps, not intersections.

This change removes confusion about using acceleration lanes at stop-controlled intersections and improves the visibility of the text by placing it in its own paragraph.

9.7 Auxiliary Lanes, 9.7.2 Deceleration Lanes (pages 9-125 to 9-127)

Proposed Revision to Green Book

Figure 9-48 [reproduced as Figure 6-1 in this report] illustrates the upstream functional area of an intersection in relation to the components of deceleration lane length, which consist of the perception-reaction (PR) distance, the lane change and deceleration distance ~~full deceleration length~~ (also called the maneuver distance), and the storage ~~length~~ distance (also called the queue storage ~~length~~ distance). ~~The physical length of a deceleration lane for turning vehicles consists of the entering taper length, L_2, the deceleration length, L_3, and the storage length, L_4.~~

Desirably, the total physical length of the auxiliary lane should be the sum of the length for these three components (lane change, deceleration, and storage distances). 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. Each component of the deceleration lane length is discussed below.

Perception-Reaction Distance

The PR distance (d_1) in Figure 9-48 represents the distance traveled while a driver recognizes the upcoming turn lane and prepares for the left-turn maneuver. It increases with perception-reaction time and speed. The perception-reaction time varies with the driver's familiarity with the roadway segment and state of alertness; for example, an alert driver who is familiar with the roadway and traffic conditions has a smaller perception-reaction time than an unfamiliar driver. Traffic conditions on urban and suburban roadways could result in drivers having a higher level of alertness than those on rural highways. Therefore, a value of 1.5 sec is often used as the perception-reaction time for urban and suburban conditions, and 2.5 sec is often used for rural situations (83).

Lane Change and Deceleration Length

On many facilities, it is not practical to provide the full length of the auxiliary lane for deceleration due to constraints such as restricted right-of-way, distance available between adjacent intersections, and extreme storage needs. In such cases, at least part of the deceleration by drivers needs to be accomplished before entering the auxiliary lane. Research (*98*) has shown that crash potential increases as the difference in speed increases between vehicles in a traffic stream. In particular, the research concludes that a vehicle traveling 10 mph slower than other traffic (i.e., a vehicle with a 10-mph speed differential) is twice as likely to be involved in, or cause, a crash than when all vehicles are traveling at the same speed; the likelihood of a crash increases exponentially with greater speed differentials. Shorter auxiliary lane lengths will increase the speed differential between turning vehicles and through traffic. Therefore, the distances shown in Table 9-22 should be provided when possible, and designers are encouraged to provide sufficient deceleration length such that drivers do not need to decelerate more than 15 km/h [10 mph] within the through lane to accommodate the design. The deceleration distances discussed 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.

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. Research (*121*) has shown that a two-stage deceleration process that uses rates of 4.2 ft/s$_2$ while the driver changes lanes into the turning lane and 6.5 ft/s$_2$ within the deceleration lane would accommodate most drivers. In locations with geometric constraints, a design that incorporates a higher deceleration rate could be offered as an alternative. While drivers may be able to negotiate the left-turn lane at higher deceleration rates, even up to the 11.2 ft/s$_2$ rate described in Section 3.2.2 for stopping sight distance, a design that accommodates lower rates provides a more conservative design that is less demanding on drivers and contains more provision for storage of queues of left-turning vehicles. A design alternative for constrained conditions can be accomplished through the use of a constant 6.5 ft/s$_2$ rate throughout the full deceleration length. Table 9-22 presents the estimated distances needed by drivers to maneuver from the through lane into the left-turn lane~~a turn bay~~ and brake to a stop ~~(6)~~; typical lengths are based on a two-stage deceleration process with rates of 4.2 ft/s$_2$ and 6.5 ft/s2, and constrained lengths are based on a constant 6.5 ft/s2 rate. Typical lengths should be used for new roadway projects and for reconstruction projects where sufficient right-of-way exists to provide the additional length. Constrained lengths may be considered at locations where the existing roadway is being changed and adjacent development prohibits the use of typical lengths.

~~On many facilities, it is not practical to provide the full length of the auxiliary lane for deceleration due to constraints such as restricted right-of-way, distance available between adjacent intersections, and extreme storage needs. In such cases, at least part of the deceleration by drivers needs to 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 15 km/h [10 mph] before clearing a through lane. Shorter auxiliary lane lengths will increase the speed differential between turning vehicles and through traffic. A 15-km/h [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 distances discussed above should be accepted as a desirable goal and should be provided where practical. The deceleration distances discussed 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.~~

Discussion

Note 1 in Table 9-22 has a typo; it should be a + sign instead of an = sign. Reference #6 above the existing Green Book Table 9-22 is an NHI course on access management from 1998 (84), but the actual content of Table 9-22 and its notes are credited to Stover and Koepke's *Transportation and Land Development* (*83*) in the TRB *Access Management Manual* (AMM) (*13*). It is unclear which of these is the correct source and whether this should this be the source for determining appropriate deceleration rates and lengths. Much of the existing material is very similar to that found in the AMM, so it is logical that the AMM's sources are also the Green Book's sources.

In contrast, the Green Book's discussion of a 10-mph difference in speed does not have the same support as the corresponding text in the AMM. The Green Book states that a 10-mph differential between turning and through vehicles is commonly considered acceptable on arterial roadways, though 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. Given that higher speed differentials also occur on high-speed arterials as found in this research, it would be useful for the Green Book to clarify on what basis that differential is "acceptable" and add the references to the research or other guidance documents that support the explanation. Referencing research by Soloman (98) or others, which describes the increased likelihood of crashes as speed differential increases above 10 mph, establishes a basis for recommending that designers provide left-turn lane designs that do not require speed differentials greater than 10 mph. This research also identified smaller speed differences for sites with longer combined taper and deceleration lengths.

The results of the research conducted as part of this study generated the recommended language changes along with the replacement Figure A-2 and Table 2-1.

~~Table 9-22. Desirable full deceleration lengths.~~

~~Metric~~		~~U.S. Customary~~	
~~Speed, km/h~~	~~Distance,[a] m~~	~~Speed [mph]~~	~~Distance[a] [ft]~~
~~30~~	~~20~~	~~[20]~~	~~[70]~~
~~50~~	~~45~~	~~[30]~~	~~[160]~~
~~65~~	~~85~~	~~[40]~~	~~[275]~~
~~80~~	~~130~~	~~[50]~~	~~[425]~~
~~95~~	~~185~~	~~[60]~~	~~[605]~~
~~110~~	~~245~~	~~[70]~~	~~[820]~~

~~[a] Rounded to 5m [5 ft]~~

~~Notes:~~

~~1. The above full deceleration lengths are L2 = L3 in Figure 9-48.~~

~~2. Assumes a turning vehicle has "cleared the through lane" when it has moved laterally approximately 3 m [9 ft] so that a following through vehicle can pass without encroaching upon the adjacent traffic lane.~~

~~3. The speed differential between the turning vehicle and following through vehicles is 15 km/h [10 mph] when the turning vehicle "clears the through traffic lane."~~

~~4. 1.8 m/s^2 [5.8 ft/s^2] deceleration while moving from the through lane into the turn lane; 2.0 m/s^2 [6.5 ft/s^2] average deceleration after completing lateral shift into the turn lane.~~

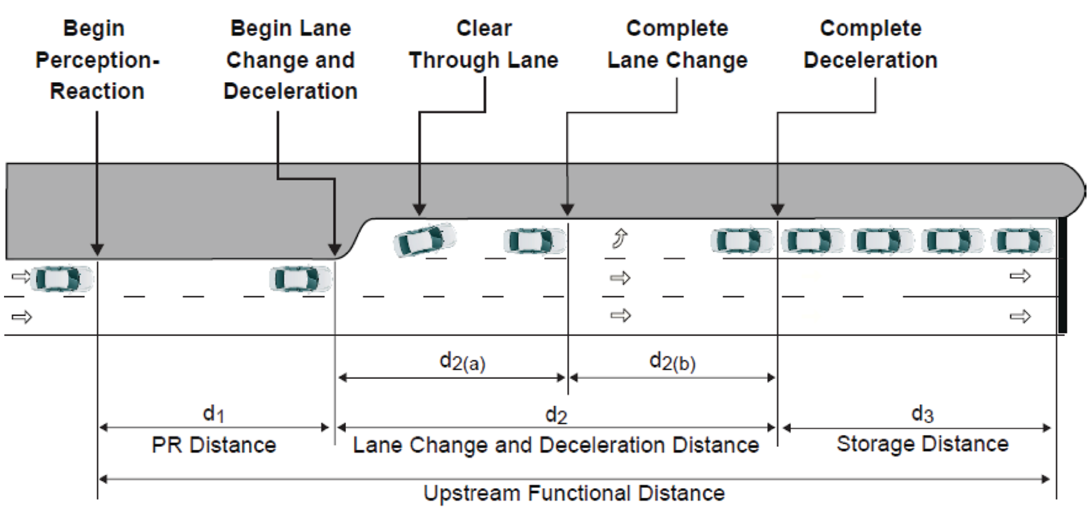
Table A-3. Example material to replace Green Book Table 9-22. Desirable lane change and deceleration distances.

Metric			U.S. Customary		
Speed, km/h	Lane Change and Deceleration Distance,[a] m		Speed [mph]	Lane Change and Deceleration Distance [a] [ft]	
	Typical	Constrained		Typical	Constrained
30	29.0	21.3	[20]	[95]	[70]
40	42.7	32.0	[25]	[140]	[105]
50	59.4	45.7	[30]	[195]	[150]
55	79.2	62.5	[35]	[260]	[205]
65	100.6	80.8	[40]	[330]	[265]
70	125.0	103.6	[45]	[410]	[340]
80	152.4	126.5	[50]	[500]	[415]
90	181.4	153.9	[55]	[595]	[505]
95	213.4	182.9	[60]	[700]	[600]
105	246.9	213.4	[65]	[810]	[700]
110	283.5	248.4	[70]	[930]	[815]

[a] Rounded to 5 ft and converted to equivalent values rounded to 0.1 m

Notes:

1. The above full deceleration lengths are $d_{2a} + d_{2b}$ in Figure 9-48.
2. The speed differential between the turning vehicle and following through vehicles is 15 km/h [10 mph] when the turning vehicle clears the through-traffic lane and completes the lane change (i.e., distance d_{2a}).
3. Deceleration lengths for a Typical installation are based on 1.3 m/s^2 [4.2 ft/s^2] deceleration while moving from the through lane into the turn lane (distance d_{2a}) and 2.0 m/s^2 [6.5 ft/s^2] deceleration after completing the lane change (distance d_{2b}).
4. Deceleration lengths for a Constrained installation are based on a 2.0 m/s^2 [6.5 ft/s^2] deceleration throughout the entire length.
5. Typical lengths should be used for new roadway projects and for reconstruction projects where sufficient right-of-way exists to provide the additional length.
6. Deceleration rates are based on deceleration on dry, level pavement. Designs for approaches on downgrades of more than 2% and intersections at locations prone to wet pavement should account for the additional length necessary for vehicles to decelerate to a stop in those conditions.
7. Access points should not be allowed in the deceleration areas.



Where:

d_1 = Distance traveled while driver recognizes upcoming turn lane and prepares for the left-turn maneuver.

$d_{2(a)}$ = Distance traveled while decelerating and changing lanes from the through lane into the turn lane.

$d_{2(b)}$ = Distance traveled during deceleration after lane change.

d_3 = Distance provided for the storage of the queue of stopped vehicles waiting to turn.

Figure A-2. Example graphic for replacing Green Book Figure 9-48. Functional area upstream of an intersection illustrating components of deceleration lane length.

9.7 Auxiliary Lanes, 9.7.2 Deceleration Lanes (pages 9-127 to 9-130)

Proposed Revision to Green Book

Taper Length

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

As shown in Figure 9-48 and Table 9-22, the physical length of a deceleration lane for turning vehicles consists of the entering taper length, the length of the full-width deceleration lane, and the storage length. The distance over which the initial deceleration occurs, though it takes place during the lane change, does not necessarily coincide exactly with the taper length. The longitudinal location along the highway where a vehicle will change lanes from the through lane to a full-width deceleration lane will vary depending on many factors. These factors include the type of vehicle, the driving characteristics of the vehicle operator, the speed of the vehicle, the number of vehicles already queued in the turn lane, weather conditions, and lighting conditions.

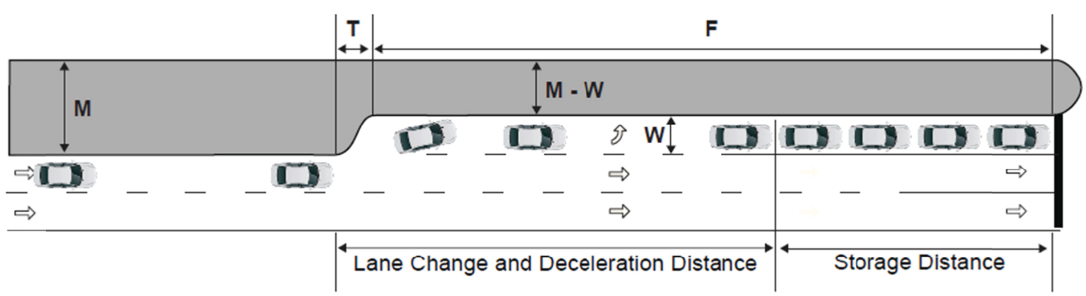
Two common methods are available for a designer to determine the actual length of the taper to be used for a deceleration lane. In the first method, the taper length is selected first, based on the guidance in the preceding paragraphs for a taper that is appropriate for the location being considered. Once the designer determines the appropriate taper rate and length, the length can then be related to Table 9-22 to determine the corresponding length of the full-width deceleration lane needed to provide the typical (or constrained) length shown in the table. Conversely, the designer may decide to first provide a specific length of full-width deceleration lane; if so, the designer would then subtract that length from the appropriate value in Table 9-22 to determine the corresponding taper length.

Drivers may complete their lane change downstream of the taper, particularly at locations with short or "squared-off" tapers. The difference between the deceleration distance and the physical boundaries of the taper and full-width deceleration lane for a left-turn auxiliary lane is displayed in Figure 9-XA; dimensions for a right-turn lane are similar to those for a left-turn lane.

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 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.~~ Short tapers also allow more length for full-width deceleration distance and do not restrict the ability of drivers to complete their lane changes further upstream of the intersection. This type of design may reduce the likelihood that entry into the auxiliary lane may spill back into the through lane. ~~Municipalities and urban counties~~ Jurisdictions across the country are increasingly adopting the use of taper lengths ~~such~~ as short as ~~30~~ 15 m [~~100~~ 50 ft] for a single-turn lane and ~~45~~ 30 m [~~150~~ 100 ft] for a dual-turn lane for urban streets.

Some agencies permit the tapered section of deceleration auxiliary lanes to be constructed in a "squared-off" section at full paving width and depth. This configuration involves a painted delineation of the taper. The abrupt squared-off beginning of deceleration exits offers improved driver commitment to the exit maneuver and also contributes to driver security because of the elimination of the unused portion of long tapers. The design involves transition of the outer or median shoulders around the squared-off beginning of the deceleration lane.

The squared-off design principle can be applied to median deceleration lanes, and it can also be used at the beginning of deceleration right-turn exit terminals when there is a single exit lane. When two or more exit lanes are used, the tapered designs discussed in Section 10.9.6 under "Speed-Change Lanes" are recommended. Additional guidance for lengths of tapers may be found in the MUTCD (*7*).



Legend:
F = Full-Width Left-Turn Lane
M = Median Width
T = Taper Length
W = Left-Turn Lane Width

Figure A-3. Example graphic for new Green Book Figure 9-XA. Key dimensions for maneuvers and physical boundaries at left-turn auxiliary lanes.

~~The longitudinal location along the highway, where a vehicle will move from the through lane to a full-width deceleration lane, will vary depending on many factors. These factors include the type of vehicle, the driving characteristics of the vehicle operator, the speed of the vehicle, weather conditions, and lighting conditions.~~

Straight-line tapers are frequently used, as shown in Figure 9-49A. The taper rate may be 8:1 [L:T] for design speeds up to 50 km/h [30 mph] and 15:1 [L:T] for design speeds of 80 km/h [50 mph] and greater. 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. A short curve is desirable at each end of long tapers as shown in Figure 9-49B, 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.

Symmetrical reverse-curve tapers are commonly used on curbed urban streets. Figure 9-49C shows a design taper with symmetrical reverse curves.

A more desirable reverse-curve taper is shown in Figure 9-49D where the turnoff curve radius is about twice that of the second curve. When 30 m [100 ft] or more in length is provided for the tapers in Figure 9-49D, tapers 1 and 2 would be suitable for low-speed operations. All the example design dimensions and configurations shown in Figure 9-49 are applicable to right-turn lanes as well as left-turn lanes.

Discussion

The Green Book states, "The taper rate may be 8:1 for design speeds up to 30 mph and 15:1 for design speeds of 50 mph and greater." This was approximately true for the sites in the deceleration study, though actual tapers varied between 6:1 and 18:1. The review of state design manuals for the interim report in this research showed that states generally recommend a straight-line taper using one of the following guidelines:

- Specific lengths, perhaps based on speed, between 60 and 180 ft. (Illinois guidelines state 135–310 ft, while Florida calls for 50 ft for all single left-turn lanes.)
- Taper rate based on speed, generally between 8:1 and 15:1.
- The values shown in the Green Book.

Even though the taper rates were similar to Green Book guidelines, some deceleration took place upstream of the taper and the lane-change maneuver was not always completed within the taper at the deceleration study sites. Emphasizing that physical boundaries of turning lanes are not necessarily the same as the maneuver boundaries for drivers will add clarity to the guidelines. In addition, encouraging designers to use shorter tapers, such as Florida's 50-ft length, facilitates longer full-width deceleration lanes and/or queue storage; it also facilitates more efficient lane-change maneuvers, removing decelerating vehicles from the through travel lane more quickly and further upstream of the intersection.

9.7 Auxiliary Lanes, 9.7.3 Design Treatments for Left-Turn Maneuvers (pages 9-131 to 9-133)

Proposed Revision to Green Book

Guidelines for ~~Design~~ Installation of Left-Turn Lanes

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 affects 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.

In 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 reduce crash frequencies and preserve capacity. The provision of left-turn lanes has been found to reduce crash rates anywhere from ~~20 to 65~~ 7 to 48% (*1*) (~~9~~). Left-turn facilities should be established on roadways where traffic volumes are high enough or crash histories are sufficient to warrant them. They are often needed to provide adequate service levels for the intersections and the various turning movements.

Warrants for Left-Turn Lanes and Bypass Lanes

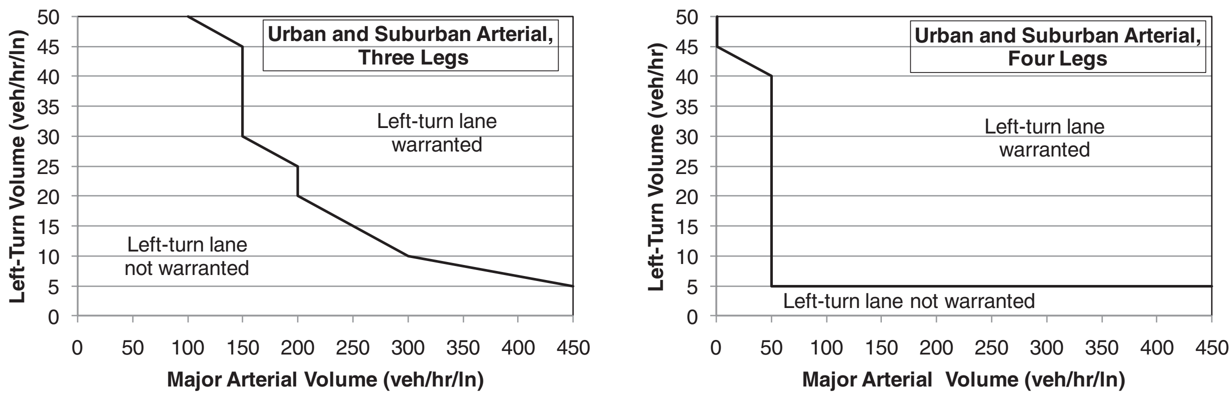
Figures 9-4B and 9-5B provide examples of bypass lanes, which are added to the outside edge of the approach, allowing through vehicles to pass left-turning vehicles on the right, while Figures 9-4C and 9-5C show traditional left-turn lanes. Regardless of the treatment, consideration of traffic demand, delay savings, crash reduction, and construction costs are all key factors in determining whether to install a left-turn lane or a bypass lane. Research on left-turn accommodations at unsignalized intersections (*9*) produced warrants for the installation of left-turn lanes and bypass lanes that account for those factors. Guidelines for where left-turn lanes should be provided are set forth in Table 9-X1 for urban and suburban roadways and Table 9-X2 and Table 9-X3 for rural highways (and Figure 9-XB, Figure 9-XC, and Figure 9-XD for urban and suburban roadways, rural two-lane highways, and rural four-lane highways, respectively). S~~s~~everal documents for both signalized and unsignalized intersections provide guidance on left-turn lanes (10, 16, 19, *9*). These guidelines discuss the need for left-turn lanes based upon (a) the number of arterial lanes, (b) design and operating speeds, (c) left-turn volumes, and (d) opposing traffic volumes.

The HCM (*29*) indicates that exclusive left-turn lanes at signalized intersections should be installed as follows:

- Exclusive left-turn lanes should be provided where exclusive left-turn signal phasing is provided;
- Exclusive left-turn lanes should be considered where left-turn volumes exceed 100 veh/h (left-turn lanes may be provided for lower volumes as well based on the highway agency's assessment of the need, the state of local practice, or both); and
- Double left-turn lanes should be considered where left-turn volumes exceed 300 veh/h.

Table A-4. Example material for new Green Book Table 9-X1. 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



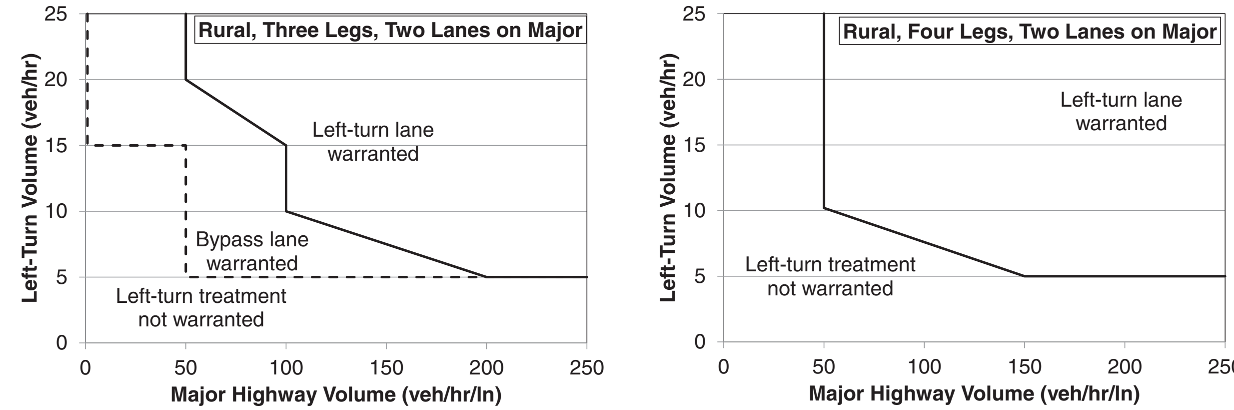
(a) Three Legs

(b) Four Legs

Figure A-4. Example graphic for new Green Book Figure 9-XB. Suggested left-turn lane warrants based on results from benefit-cost evaluations for intersections on urban and suburban arterials.

Table A-5. Example material for new Green Book Table 9-X2. 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 Left-Turn Lane
5	50	200	150
10	50	100	50
15	≤ 50	100	50
20	≤ 50	50	≤ 50
25	≤ 50	50	≤ 50
30	≤ 50	50	≤ 50
35	≤ 50	50	≤ 50
40	≤ 50	50	≤ 50
45	≤ 50	50	≤ 50
50 or More	≤ 50	50	≤ 50



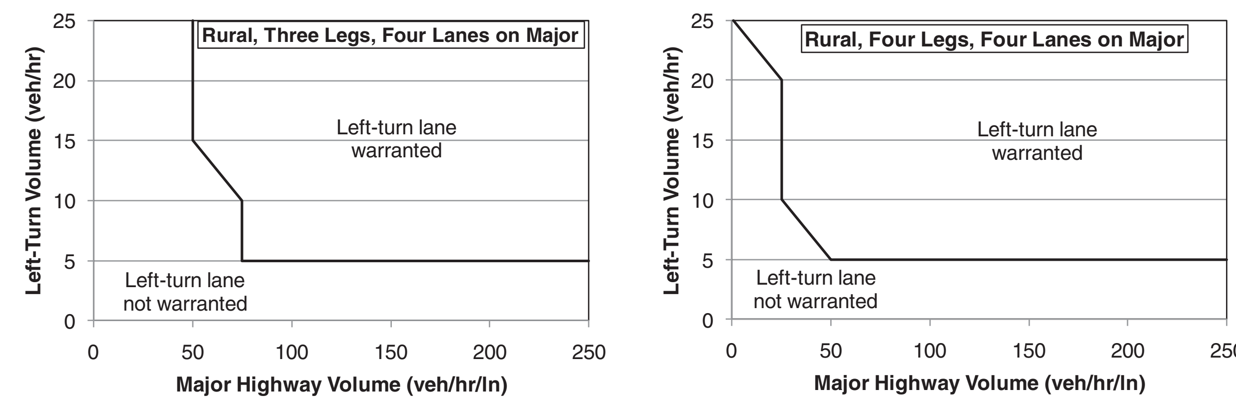
(a) Three Legs

(b) Four Legs

Figure A-5. Example graphic for new Green Book Figure 9-XC. Suggested left-turn treatment warrants based on results from benefit-cost evaluations for intersections on rural two-lane highways.

Table A-6. Example material for new Green Book Table 9-X3. 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-6. Example graphic for new Green Book Figure 9-XD. Suggested left-turn lane warrants based on results from benefit-cost evaluations for intersections on rural four-lane highways.

~~Table 9-23. Guide for left-turn lanes on two-lane highways (10)~~

~~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~~

~~Table 9-23 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 ~~Highway Research Record 211,~~ NCHRP ~~Report~~ Synthesis 225, ~~and~~ *NCHRP Report 279*, and NCHRP Report 745 (~~10~~, 19, 17, *9*). 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.

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~~ predominant highway, is adequate to serve normal traffic plus the traffic diverted from any terminated road or street.

Discussion

Much of the subsection on "Guidelines for Design of Left-Turn Lanes" is really more about the installation of left-turn lanes. This subsection could be split into multiple parts, to include discussion of installation, warrants, and general design principles. The existing text in the first two paragraphs is on installation. The suggested revised text can be presented as a subsection on warrants to fill an information need. The remaining text can then be the subsection on "Guidelines for Design of Left-Turn Lanes."

Also, there is a minor typo in the last paragraph that begins on page 9-132.

The revisions add the warrant information developed in NCHRP Project 3-91 (NCHRP Report 745) (*9*) and correct some typographical errors in the text. The suggested revised text is from Appendix A of NCHRP Report 745, updated to reflect changes between the 2004 Green Book used as the source then and the 2011 edition used now.

9.7 Auxiliary Lanes, 9.7.3 Design Treatments for Left-Turn Maneuvers (page 9-137)

Proposed Revision to Green Book

Offset Left-Turn Lanes

For medians wider than about 5.4 m [18 ft], it is desirable to offset the left-turn lane so that it will reduce the width of the divider to 1.8 to 2.4 m [6 to 8 ft] immediately before the intersection, rather than to align it exactly parallel with and adjacent to the through lane. This alignment will place 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 (1) better visibility of opposing through traffic; (2) decreased possibility of conflict between opposing left-turn movements within the intersection; and (3) more left-turn vehicles served in a given period of time, particularly at a signalized intersection (*122*). Figure 9-XE provides suggested minimum widths for separator and offset islands for different conditions.

Parallel offset left-turn lanes may be used at both signalized and unsignalized intersections. This left-turn lane configuration is referred to as a parallel offset left-turn lane and is illustrated in Figure 9-52A.

Discussion

The draft Access Management Manual (*97*) includes a graphic (see Figure A-7) showing minimum width for the offset island. This information could be added to the Green Book to provide additional guidance on offset dimensions.

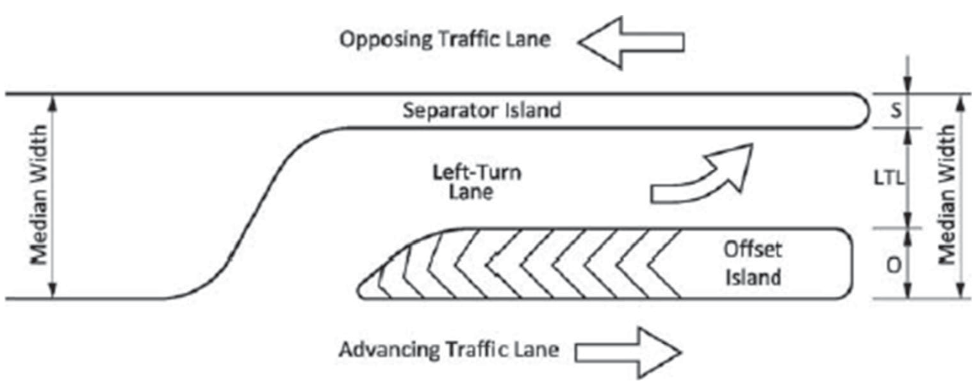
9.7 Auxiliary Lanes, 9.7.3 Design Treatments for Left-Turn Maneuvers (page 9-139)

Proposed Revision to Green Book

Double or Triple Left-Turn Lanes

Where two median lanes are provided as a double left-turn lane, left-turning vehicles leave the through lanes to enter the median lanes in single file, but once within the median lanes, the vehicles are stored in two lanes. On receiving the green indication, the left-turning vehicles turn simultaneously from both lanes.

~~With three-phase signal control, such an arrangement results in~~ Research (*121*) has shown an increase in capacity for double left-turn lanes of approximately ~~180~~ 195% of that of a single median lane. Occasionally, the two-abreast turning maneuvers may lead to sideswipe crashes. These usually result from too sharp a turning radius or a roadway that is too narrow. The receiving leg of the intersection should have adequate width to accommodate two lanes of turning traffic. A width of 9 m [30 ft] is used by several highway agencies. Capacity benefits can be achieved if the receiving leg width is greater than 36 ft; however, the tradeoff for a wider crossing distance and increase in costs are to be considered. Capacity benefits for the left-turn operation on the order of 56 pcphgpl for each U-turning vehicle will also be achieved if U-turns are prohibited for the double left-turn maneuver. ~~Triple~~



Condition	S Separator Island (feet)	LTL Left-Turn Lane (feet)	O Offset Island (feet)	Minimum Median Width (feet)
Standard Design:				
Pedestrians	≥ 6	12	≥ 3.5	≥ 21.5
No Pedestrians	≥ 4	12	≥ 3.5	≥ 19.5
Permitted by Variance:				
Pedestrians	≥ 6	12	≥ 2	≥ 20
No Pedestrians	≥ 4	12	≥ 2	≥ 18
No Pedestrians	≥ 3	11	≥ 2	≥ 16

Source: *Access Management Manual* (draft second edition), exhibit 17-7. Reproduced with permission of the Transportation Research Board.

Figure A-7. Example graphic for new Green Book Figure 9-XE. Minimum width with offset island.

~~left-turn lanes have also been used at locations with very high left-turn volumes. Double and triple turning lanes should only be used with signalization providing a separate turning phase.~~

Multiple left-turn lanes are becoming more widely used at signalized intersections where traffic volumes have increased beyond the design volume of the original single left-turn lane. The following are design considerations for double or triple left-turn lanes:

- Width of receiving leg.
- Width of intersection (to accommodate the two or three vehicles turning abreast).
- Clearance between opposing left-turn movements if concurrent maneuvers are used.
- Turning path width for design vehicle.
- Pavement marking visibility.
- Location of downstream conflict points.
- Weaving movements downstream of turn.
- Potential for pedestrian conflict.

Offtracking and swept path width are important factors in designing double and triple left-turn lanes. At such locations, vehicles should be able to turn side by side without encroaching upon the adjacent turn lane. A desirable turning radius for a double or triple left-turn lane is 27 m [90 ft], which will accommodate the P, SU-9 [SU-30], SU-12 [SU-40], and WB-12 [WB-40] design vehicles within a swept path width of 3.6 m [12 ft]. Larger vehicles need greater widths to negotiate double or triple left-turn lanes constructed with a 27 m [90 ft] turning radius without encroaching on the paths of vehicles in the adjacent lane.

Table 9-24 illustrates the swept path widths for specific design vehicles making 90-degree left turns (*51*). Table 9-24 can be used to determine the width needed at the center of a turn where the maximum vehicle offtracking typically occurs.

To help drivers maintain their vehicles within the proper lanes, the longitudinal lane line markings of double or triple left-turn lanes may be extended through the intersection area to provide positive guidance (see MUTCD Section 3B.08, Extensions Through Intersections or Interchanges for guidance). This type of pavement marking extension is intended to provide a visual cue for lateral positioning of the vehicle as the driver makes a turning maneuver.

Discussion

The findings from this project's double left-turn lane study (*121*) generated the following potential recommendations:

- The Green Book states that the capacity of double left-turn lanes is approximately 180% of that of a single median lane. Per the *Highway Capacity Manual* (*8*) the base saturation flow rate for a metropolitan area with population of 250,000 is 1900 pcphgpl and the left-turn adjustment factor is 1/1.05. Comparing the single-lane saturation flow rate (1900/1.05 = 1810 pcphgpl) to the average saturation flow rates for the double left-turn lanes sites in this study (1774 + 1776 = 3550 pcphgpl) results in a value (3550/1810 = 1.96 or 196 percent) that is greater than 180 percent.
- The Green Book states that the receiving leg of the intersection should have adequate width to accommodate two lanes of turning traffic and that a width of 30 ft is used by several highway agencies. Early literature by Neuman (*5*) stated the throat width for the turning traffic is the most important design element, and that because of the offtracking characteristics of vehicles, a 36-ft throat width is desirable for acceptance of two lanes of turning traffic. In constrained situations, 30-ft throat widths are acceptable minimums. Within this study, the pattern of increasing saturation flow rate for increasing receiving leg width was examined to try to identify if there were dimensions where a sizable increase in saturation flow rate occurs. The method concluded that the change point occurs between receiving leg widths of 36 ft and 40 ft.
- Additional discussions or cautions in the section on multiple turn lanes:
  - Double left-turn vehicles, turning into a receiving leg of two lanes where a third lane is being added as a dedicated downstream lane from a channelized right-turn lane, were observed to move into the additional lane as soon as physically possible, even across a solid white line.
  - The number of U-turning vehicles has a significant impact on the operations of double left-turn lanes.

The text in the Green Book was revised to reflect the higher saturation flow rate (rounded the 196% to 195 percent) and to note the potential increase in capacity available by using a wider leg width and prohibiting U-turns.

Findings from this research project (121), previous literature (7, 52, 73, 74), and the engineering judgment of the research team developed the list of design considerations for double or triple left-turn lanes.

9.8 Median Openings, 9.8.1 General Design Considerations (page 9-140)

Proposed Revision to Green Book

Medians are discussed in Section 4.11 chiefly as an element of the cross-section. General ranges in width are given, and median width at intersection is treated briefly. For intersection conditions, the median width, the location and length of the opening, and the design of the median end are developed in combination to fit the character and volume of through and turning traffic. Figure 9-50 illustrates the appropriate dimensions for the median width and the length of median opening. Median openings should reflect street or block spacing and the access classification of the roadway. In addition, full median opening should be consistent with traffic signal spacing criteria. In some situations, median openings should be eliminated or made directional.

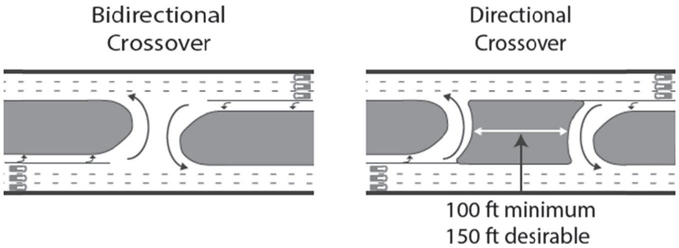


Figure A-8. Example graphic for new Green Book Figure 9-XF. Bidirectional (conventional) and directional median openings.

When considering construction or elimination of a median opening, the following guidance can be considered:

- For median widths less than 9 m [30 ft] in width, median openings should be constructed opposite driveways and crossroads as appropriate.
- For median widths greater than 9 m [30 ft] in width, median openings should be constructed every 400 m [1320 ft] in urban areas and 800 m [2640 ft] in rural areas. This spacing may be adjusted by 30 m [100 ft] either way to conform to driveways and crossroads. No two median opening should be closer than 150 m [500 ft] apart unless servicing a through crossroad.

There are two types of median crossover intersections: bidirectional (sometimes called conventional) and directional (see Figure 9-XF). A bidirectional crossover allows vehicles to make a U-turn from either direction of travel, which creates additional points of conflict as compared to the directional crossover. Further, as turning volumes increase, an interlocking of travel paths can occur in a bidirectional crossover, which could limit sight distance and result in unpredictable driver behavior. Several studies have shown that crash rates at directional crossovers are less than bidirectional crossovers for signalized corridors and that directional crossovers provide better operational performance.

If the median is wide enough to permit storage of vehicles, the use of a centerline and stop bar in the median storage area can communicate to drivers how the median should be negotiated and provide a sense of storage area. This can reduce undesirable maneuvers such as side-by-side queuing and lane encroachment. Further, it can communicate that it is allowable to make crossing and turning maneuvers in stages at this intersection.

Discussion

There is no guidance listed for under which conditions a median opening should be eliminated. Further, there is no guidance given on when it should be made directional. This is compounded by the fact that there is no discussion of the difference between a bidirectional (conventional) crossover and a directional crossover. Additional sources for information on median openings are in the following: Michigan DOT *Geometric Design Guide 670* (*102*), Michigan DOT *Road Design Manual* (*103*), FHWA *Alternative Intersections/Interchanges: Informational Report* (*104*), and NCHRP Report 650 (*105*).

There is no discussion of the impact of providing markings or a divisional island for vehicle storage in the median. Guidance regarding the impact of proper pavement markings on driver behavior should be added as available in NCHRP Report 650 (*105*).

9.8 Median Openings, 9.8.2 Control Radii for Minimum Turning Paths (page 9-144)

Proposed Revision to Green Book

The customary intersection on a divided highway does not have a continuous physical edge of traveled way delineating the left-turn path. Instead, the driver has guides at the beginning and at the end of the left-turn operation: (1) the centerline of an undivided crossroad of the median edge of a divided crossroad, and (2) the curved median end. For the central part of the turn the driver has the open central intersection area in which to maneuver. Under these circumstances for minimum design of the median end, the precision of compound curves does not appear to be needed, and simple curves for the minimum assumed edge of left turn have been found satisfactory. The larger the simple curve radius used, the better it will accommodate a given design vehicle, but the resulting layout for the larger curve radius will have a greater length of median opening and greater paved areas than one for a minimum radius, These areas may be sufficiently large to result in erratic maneuvering by small vehicles, which may interfere with other traffic. To reduce the effective size of the intersection for most motorists, consideration should be given to providing an edge marking corresponding to the desired turning path for passenger cars, while providing sufficient paved area to accommodate the turning path of an occasional large vehicle. Minimizing the median opening length has shown to improve intersection safety performance for rural divided highways; as such, the minimum turning radius for the design vehicle should be used for design purposes.

Discussion

This section should also emphasize the safety impacts of minimizing the median opening length. Much of the discussion focuses on curve radii for the median design, but forgets that the overall length of the opening is important as well. NCHRP Report 650 provides discussion regarding median design (*105*).

9.8 Median Openings, 9.8.3 Minimum Length of Median Opening (page 9-149)

Proposed Revision to Green Book

The use of a minimum length of opening without regard to the width of median or the control radius should not be considered except at very minor crossroads. ~~Care should be taken not to make~~ The median opening should not be longer than needed at rural unsignalized intersections to reduce undesirable median maneuvers. The minimum length of opening for U-turns is discussed in Section 9.9 on "Indirect Left Turns and U-Turns."

Discussion

The phrase "care should be taken" is awkward and vague and the second sentence should be reworded to eliminate it. The statement that "care should be taken not to make the median opening longer than needed" should be expanded to give the reason for the guidance. Further, a reference to NCHRP Report 650 (*105*) should be considered here so that users can find the reference material for the statement.

9.9 Indirect Left Turns and U-Turns, 9.9.1 General Design Considerations (pages 9-155 to 9-157)

Discussion

There is an ongoing research effort that may become a useful resource for alternative intersection design. This guidebook is due to be completed in December 2013. As such, a placeholder is being added as a reminder to review this resource when it becomes available and determine if it is something that should be added as a reference resource to this section.

- *Alternative Intersections Comparative Analysis*, Rakha, Hesham, Virginia Polytechnic Institute, completion date December 2013. The purpose of this project is to develop a guidebook for the analysis of alternative intersection designs to be used by consulting engineers performing comparative analysis during preliminary engineering. Intersections considered: displaced left-turn intersection, median U-turn intersection, restricted crossing U-turn intersection, quadrant roadway intersection, jughandle intersection, and modern roundabout.

9.9 Indirect Left Turns and U-Turns, 9.9.2 Intersections with Jughandle or Loop Roadways (pages 9-157 to 9-158)

Proposed Revision to Green Book

> Jughandles are one-way roadways in two quadrants of the intersection that allow for removal of left-turning traffic from the through stream without providing left-turn lanes. All turns—right, left, and U-turns—are made from the right side of the roadway. Drivers wishing to turn left exit the major roadway on the right and turn left onto the minor road at a terminus separated from the main intersection. There should be 30 m [100 ft] between the jughandle intersection and the stop bar for the primary intersection. Less right-of-way is needed along the roadway because the left-turn lanes are not needed. However, more right-of-way is needed at the intersection to accommodate the jughandles. Each jughandle typically requires a triangle 120 m [400 ft] by 90 m [300 ft] in the quadrant deployed. Figure 9-60 illustrates a jughandle intersection with the diagonal connecting roadways in advance of the intersection. The possible movements are illustrated in Figure 9-61.
>
> . . .
>
> Table 9-X4 shows the number of conflict points at a four-leg signalized intersection as compared to a four-leg signalized intersection with two jughandles. The use of the jughandles reduces the crossing (left-turn) conflict points by 6.

Discussion

There is no guidance given on the spacing of jughandle intersection and the primary intersection. Further, there is no sense of the space that this type of intersection takes up. References that discuss jughandles include the following: FHWA, *Alternative Intersections/Interchanges: Informational Report* (*104*), FHWA, *Signalized Intersections: Informational Guide* (*7*), Hummer et al. (*123*), and New Jersey DOT *Roadway Design Manual* (*124*).

Figure 9-60 needs to be improved, as there are several issues with it as presented. First, the jughandle roadways should intersect the cross road at as near to 90 degrees as possible. Second, there should be a short description of the conflict points associated with the intersection type as compared to a standard intersection. The suggested figure shown in Figure A-9 is given as a guide to replace Figure 9-60 (shown in Figure A-10).

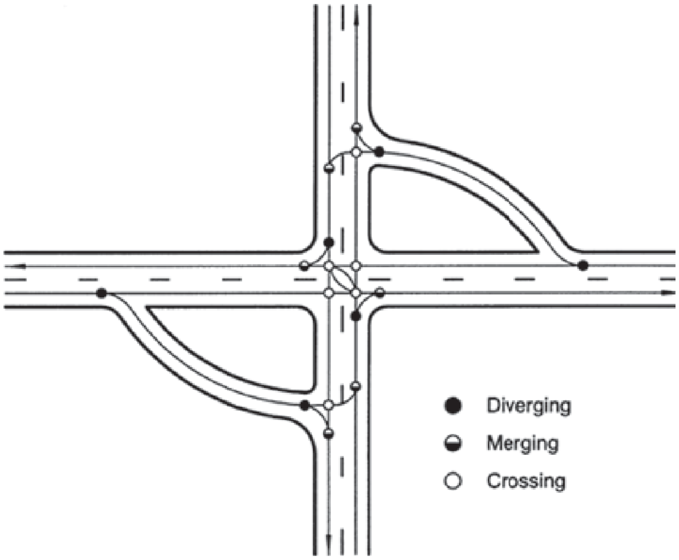
9.9 Indirect Left Turns and U-Turns, 9.9.2 Intersections with Jughandle or Loop Roadways (page 9-159)

Proposed Revision to Green Book

> An alternative to providing a jughandle ramp in advance of the intersection is to provide a loop roadway beyond the intersection. The loop design may be considered when the right-of-way

Table A-7. Example material for new Green Book Table 9-X4. Number of conflict points at a four-leg signalized intersection compared to a four-leg signalized intersection with two jughandles.

Conflict type	Four-Leg Signalized Intersection	Four-Leg Signalized Intersection with Two Jughandles
Merging/diverging	16	16
Crossing (left turn)	12	6
Crossing (angle)	4	4
Total	32	26



Source: FHWA *Signalized Intersections: Informational Guide* (*7*)

Figure A-9. Example graphic for replacing Green Book Figure 9-60. Intersection with jughandle roadways for indirect left turns.

for the ~~farside~~ far-side quadrant is less expensive than that for the ~~nearside~~ near-side quadrant. Vertical alignment and comparative grading costs may also influence the intersection quadrant where the turning roadway is placed. The left-turn movement becomes a right-turn movement at the ~~intersection of a~~ far-side loop roadway ~~with the crossroad~~, resulting in fewer conflicts and higher capacity for the left-turn movement. However, this increases the intersection's entering volume and doubles the exposure of drivers at the primary intersection as drivers making this maneuver must navigate the primary intersection twice. ~~If a right-turn lane is provided on the near side of the intersection, left-turn movements from the main roadway are eliminated.~~ Figure 9-62 illustrates the use of a loop roadway beyond the intersection.

Discussion

Within this paragraph both farside (one word) and far-side (with hyphen) is used. Also nearside (one word) and near side (two words) is used. These inconsistencies should be repaired. Further, wording of the description of where the left-turn maneuver is made is confusing. Additionally, the second to the last sentence does not make sense in this context.

There is no discussion of one of the critical weaknesses of the far-side jughandle intersection, which is that drivers making that movement must navigate the primary intersection twice. This increases the intersection's entering volume and doubles the exposure of drivers making this maneuver to potential collisions within the primary intersection. Potential references on jughandles are listed above.

Figure 9-62 (reproduced in this document as Figure A-11) needs improvement, as there are several issues with it as presented. First, the jughandle roadways should better show state-of-the-art for employing free-flow turns. Second, there should be a better representation of the loop roadway along with typical minimum design values. Figure A-12 shows an example of a better figure for the Green Book Figure 9-62.

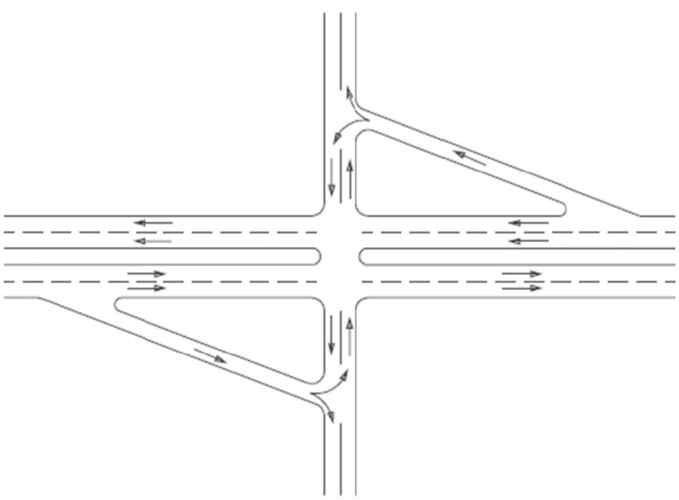


Figure 9-60. Intersection with Jughandle Roadways for Indirect Left Turns

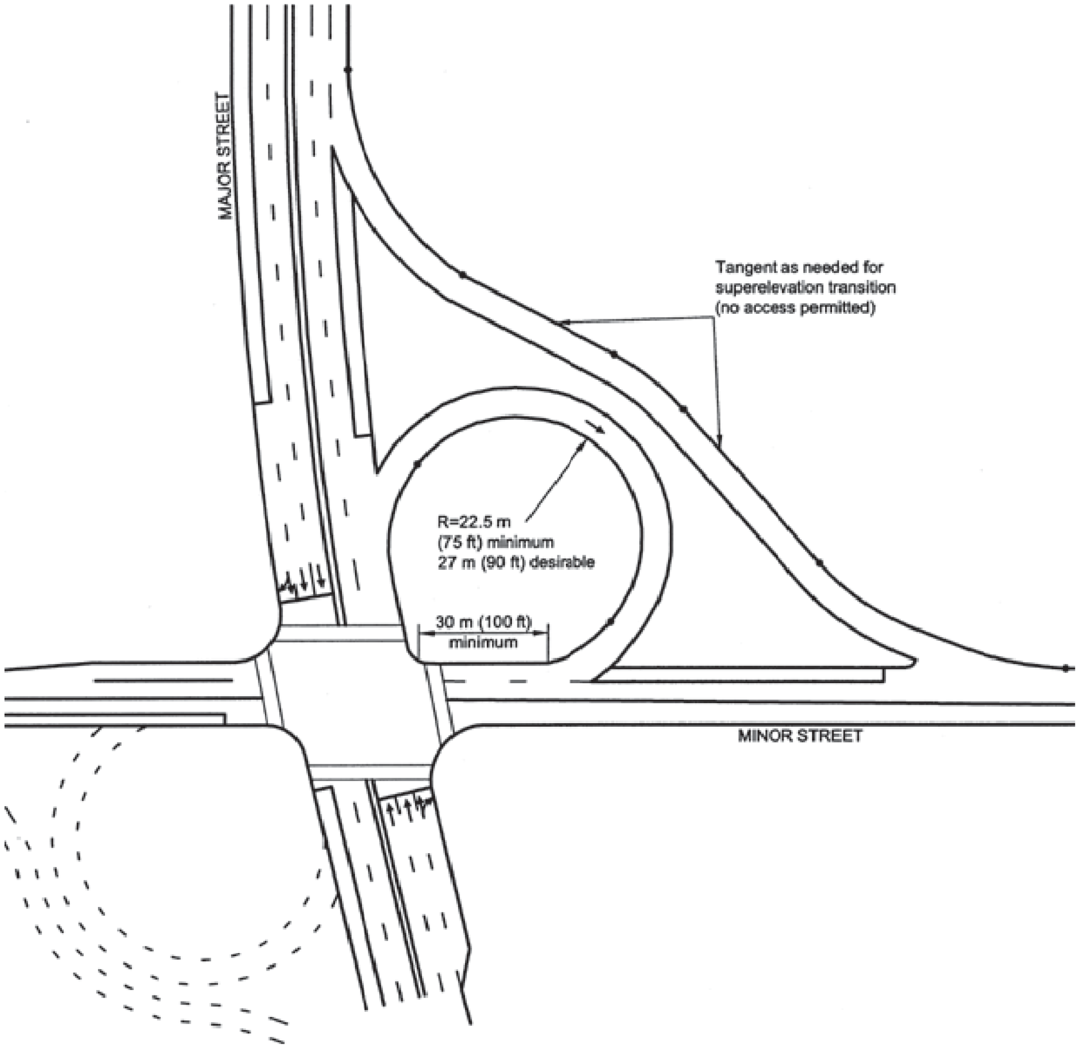
Source: *A Policy on Geometric Design of Highways and Streets* (2011) by the American Association of State Highway and Transportation Officials, Washington, D.C. Used by permission.

Figure A-10. Copy of Green Book Figure 9-60. Intersection with jughandle roadways for indirect left turns.

Figure 9-62. Intersection with Loop Roadways for Indirect Left Turns

Source: *A Policy on Geometric Design of Highways and Streets* (2011) by the American Association of State Highway and Transportation Officials, Washington, D.C. Used by permission.

Figure A-11. Copy of Green Book Figure 9-62. Intersection with loop roadways for indirect left turns.



Source: New Jersey DOT *Highway Design Manual* (*125*)

Figure A-12. Example graphic for replacing Green Book Figure 9-62. Intersection with loop roadways for indirect left turns.

9.9 Indirect Left Turns and U-Turns, 9.9.2 Intersections with Jughandle or Loop Roadways (page 9-160)

Proposed Revision to Green Book

Additional information regarding design and operation of jughandle intersections is presented in *Signalized Intersections: Information Guide (23)* and *Alternative Intersections/Interchanges: Informational Report (104).*

Discussion

An additional source of information including some typical dimensions is in the Alternative Intersections/Interchanges: Informational Report (104). Other potential references on jughandles are listed above.

9.9 Indirect Left Turns and U-Turns, 9.9.3 Displaced Left-Turn Intersections (page 9-160)

Proposed Revision to Green Book

A displaced left-turn intersection, also known as a continuous-flow intersection (CFI) or a crossover-displaced left-turn (XDL) intersection, removes the conflict between left-turning vehicles and oncoming traffic at the main intersection by introducing a left-turn bay placed to the left of oncoming traffic. Vehicles access the left-turn bay at a midblock signalized intersection on the approach where continuous flow is desired. Figure 9-63 shows the design of an intersection with displaced left-turn roadways and Figure 9-64 illustrates some of the vehicle movements at such an intersection. Table 9-X5 shows the number of conflict points at a four-leg signalized intersection as compared to a CFI with displaced

Table A-8. Example material for new Green Book Table 9-X5. Number of conflict points at a four-leg signalized intersection compared to a continuous-flow intersection with displaced left turns on the major street only.

Conflict type	Four-Leg Signalized Intersection	Continuous-Flow Intersection
Merging/diverging	16	14
Crossing (left turn)	12	6
Crossing (angle)	4	10
Total	32	30

left turns on the major street only. The use of the CFI separates the conflict points for left-turning traffic from the primary intersection.

Discussion

A discussion regarding conflict points (number and type) for CFI versus standard intersection would be helpful. A potential source of information and typical dimensions is available in the FHWA publication, *Alternative Intersections/ Interchanges: Informational Report* (*104*).

9.9 Indirect Left Turns and U-Turns, 9.9.3 Displaced Left-Turn Intersections (page 9-161)

Proposed Revision to Green Book

Additional information regarding the operation of displaced left-turn intersections is presented in *Signalized Intersections: Informational Guide (23)* and Alternative Intersections/Interchanges: Informational Report (104).

Discussion

A potential source of information and typical dimensions is available in the FHWA publication, Alternative Intersections/ Interchanges: Informational Report (*104*).

9.9 Indirect Left Turns and U-Turns, 9.9.4 Wide Medians with U-Turn Crossover Roadways (page 9-162)

Proposed Revision to Green Book

Figure 9-65 illustrates an indirect left turn for two arterials where left turns are heavy on both roads. The north-south roadway is undivided and the east-west roadway is divided with a wide median. Because left turns from the north-south road would cause congestion because of the lack of storage, left turns from the north-south road are prohibited at the main intersection. Left-turning traffic turns right onto the divided road and then makes a U-turn at a one-way crossover in the median of the divided road. Auxiliary lanes are highly desirable for the left-turn movements and the right-turn movements needed for the median U-turn operation. Figure 9-66 illustrates some of the vehicle movements at such an intersection.

Figure 9-XG illustrates a variation of this design called a restricted crossing U-turn (RCUT) intersection, also called a Superstreet intersection or a J-turn intersection. The RCUT redirects both left-turn and through movements from the crossroad to a one-way crossover in the median of the divided roadway. Left-turning traffic from the major, divided roadway still uses the primary intersection. This configuration is generally suited to higher-volume major roads in suburban and rural areas where relatively low traffic volumes enter from the crossroad. Pedestrians cross the divided roadway in a diagonal fashion, going from one corner to the opposite corner between the channelized left turns for the divided roadway. A further variant of this approach also prohibits left turns from the divided roadway at the primary intersection directing that movement to the crossovers.

Discussion

Figure 9-65 (reproduced as Figure A-13) is a poor representation of the design. The figure should be narrowed and elongated such as the suggested figure shown in Figure A-14.

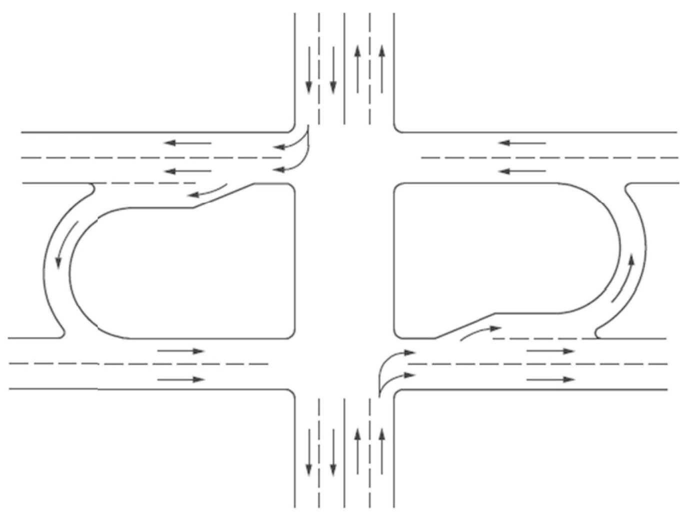


Figure 9-65. Typical Arrangement of U-Turn Roadways for Indirect Left Turns on Arterials with Wide Medians

Source: *A Policy on Geometric Design of Highways and Streets* (2011) by the American Association of State Highway and Transportation Officials, Washington, D.C. Used by permission.

Figure A-13. Copy of Green Book Figure 9-65. Typical Arrangement of U-turn roadways for indirect left turns on arterials with wide medians.

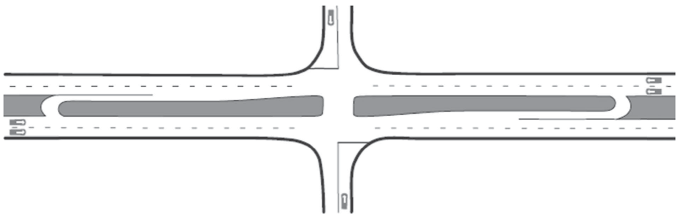


Figure A-14. Example graphic for replacement of Green Book Figure 9-65.

A modification to the median U-turn intersection is the restricted crossing U-turn intersection. A figure showing the RCUT, such as Figure A-15, should be added along with a brief description. The proposed revision is shown as a modest addition to this section, but another approach would be to create a separate section for the median U-turn intersection and another for the restricted crossing U-turn intersection. Potential sources for information are available in the following: FHWA *Alternative Intersections/Interchanges: Informational Report* (*104*), FHWA *Signalized Intersections: Informational Guide* (*7*), and Hummer et al. (*123*).

9.9 Indirect Left Turns and U-Turns, 9.9.4 Wide Medians with U-Turn Crossover Roadways (page 9-163)

Proposed Revision to Green Book

Due to their design, median U-turn crossovers need a wide median to enable the U-turn movement. For median widths of less than 20 m [60 ft], expanded paved aprons, often called "loons," can be provided opposite a median crossover. Median U-turn roadways may be appropriate at intersections with high major-street through movements, low-to-medium left turns from the major street, low-to-medium left turns from the minor street and any amount of minor street through volumes. Locations with high left-turning volumes may not be good candidates because the out-of-direction travel incurred and the potential for queue spillback at the median U-turn roadway location could outweigh the benefits associated with removing left turns from the main intersection. Median U-turn roadways can be applied on a single approach or multiple approaches.

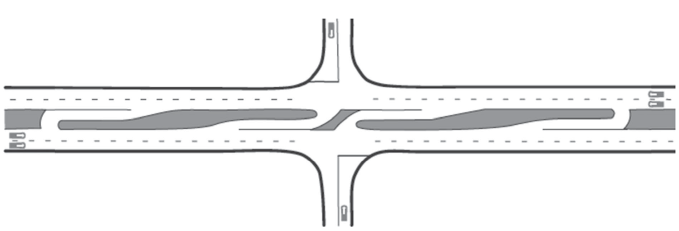


Figure A-15. Example graphic for new Green Book Figure 9-XG. Typical restricted crossing U-turn intersection.

Discussion

A comment should be added about using loons to accommodate larger vehicle U-turns with narrow medians.

9.9 Indirect Left Turns and U-Turns, 9.9.4 Wide Medians with U-Turn Crossover Roadways (page 9-164)

Proposed Revision to Green Book

Use of a median U-turn crossover intersection may result in fewer left-turn collisions and a minor reduction in merging and diverging collisions. There is a potential reduction in overall travel time and stops for mainline through movements. Findings are mixed with respect to overall stops. ~~While the number of conflicting movement at the intersections is reduced,~~ The distance for pedestrians to cross is increased and turning paths of vehicles making median U-turns may encroach into bike lanes. Additional right-of-way and access may need to be restricted within the influence of the median U-turn locations. Signing, visual cues, education, and enforcement may be needed to guide drivers to the intended turning path and minimize illegal turns.

Table 9-X6 shows the number of conflict points at a four-leg signalized intersection as compared to a four-leg signalized intersection with a median U-turn configuration. The use of the median U-turn crossover configuration eliminates all left-turn crossing conflicts while also reducing the number of merging/diverging conflict points by 4. Typical crash reductions when compared to conventional intersections range from 20 to 50 percent. The table also shows the conflict points for a restricted crossing U-turn configuration. The number of left-turn crossing conflicts is reduced to two and there are no angle crossing conflict points, though merging/diverging conflicts increase by two. Figures 9-XH and 9-XI provide conflict diagrams for the two intersection designs.

Discussion

A discussion of the conflict points for both a median U-turn intersection and a restricted crossing U-turn intersection should be added. Potential sources for additional information are available in: FHWA *Alternative Intersections/Interchanges: Informational Report* (*104*), FHWA *Signalized Intersections: Informational Guide* (*7*), and Hummer et al. (*123*).

9.9 Indirect Left Turns and U-Turns, 9.9.4 Wide Medians with U-Turn Crossover Roadways (page 9-164)

Proposed Revision to Green Book

Additional information regarding design and operation of intersections with median U-turn crossover roadways is contained in *Signalized Intersections: Informational Guide (23)*

Table A-9. Example material for new Green Book Table 9-X6. Number of conflict points at a four-leg signalized intersection compared to a four-leg signalized intersection with a median U-turn configuration and four-leg signalized intersection with a restricted crossing U-turn configuration.

Conflict type	Four-Leg Signalized Intersection	Median U-Turn Configuration	Restricted Crossing U-Turn Configuration
Merging/diverging	16	12	18
Crossing (left turn)	12	0	2
Crossing (angle)	4	4	0
Total	32	16	20

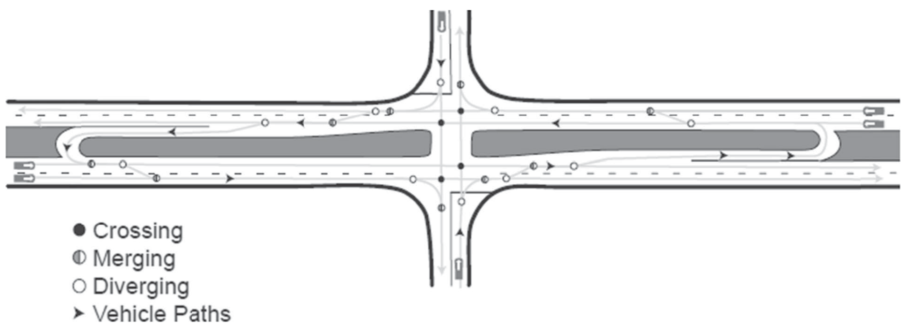


Figure A-16. Example graphic for new Green Book Figure 9-XH. Conflict diagram for median U-turn configuration.

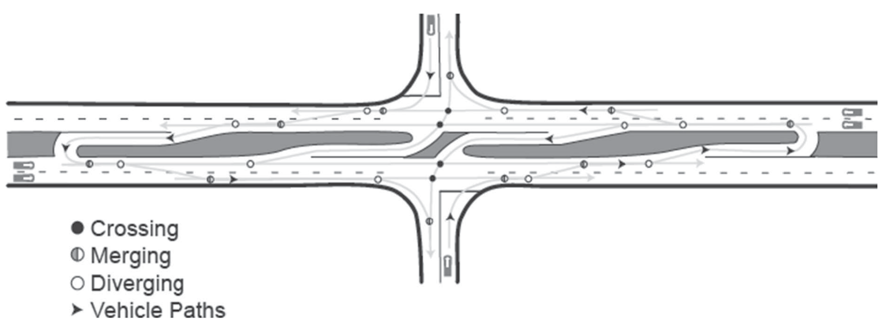


Figure A-17. Example graphic for new Green Book Figure 9-XI. Conflict diagram for restricted crossing U-turn configuration.

and *Alternative Intersections/Interchanges: Informational Report (104).* The location and design of median U-turn roadways is addressed in greater detail in Section 9.9.5.

Discussion

An additional source of information including some typical dimensions is the Alternative Intersections/Interchanges: Informational Report (104).

9.9 Indirect Left Turns and U-Turns, 9.9.5 Location and Design of U-Turn Median Openings (pages 9-165 and 9-166)

Proposed Revision to Green Book

Medians of ~~5.0~~ 5.5 m [~~16~~18 ft] and 15.6 m [~~50~~51 ft] or wider are needed to permit passenger and single-unit truck traffic, respectively, to turn from the inner lane (next to the median) on one roadway to the outer lane of a two-lane opposing roadway. Also, a median left-turn lane is highly desirable in advance of the U-turn opening to eliminate stopping on the through lanes. This scheme would increase the median width by approximately 3.6 m [12 ft].

{Within Table 9-30 (Minimum Designs for U-Turns), the values on the Metric portion should be revised to show one place past the decimal (e.g. Passenger for Inner Lane to Outer Lane should be 5.5 not simply 5).}

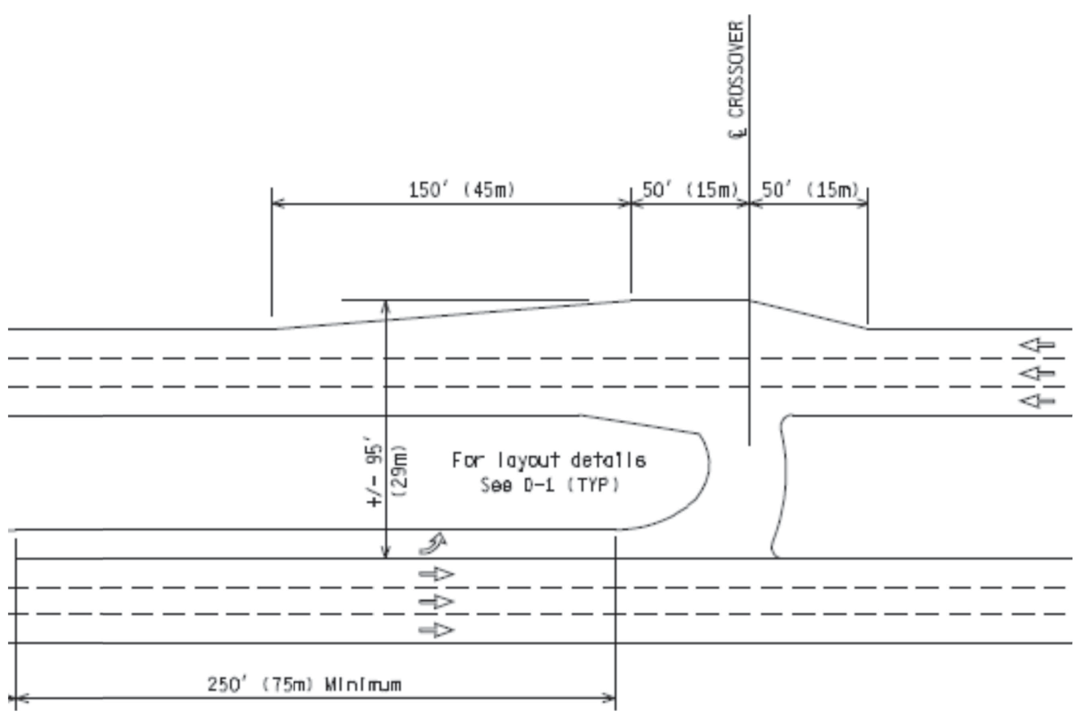
Discussion

The Green Book states "Medians of 5.0 m [16 ft] and 15 m [50 ft] or wider are needed to permit passenger and single-unit truck traffic, respectively, to turn from the inner lane (next to the median) on one roadway to the outer lane of a two-lane opposing roadway." However, Table 9-30 (p. 9-166) shows values of 5 m and 18 ft for passenger and 15 m and 51 ft for SU-30. This does not concur with the text. It appears that the decimal portion of the values listed for the metric chart were dropped (not rounded) from the table resulting in this discrepancy. As such, it is suggested that the text be corrected to read "5.5 m [18 ft] and 15.6 m [51 ft]" to address this discrepancy. Further, Table 9-30 Metric should be updated to include the decimal portion of the result so that the two tables agree.

9.9 Indirect Left Turns and U-Turns, 9.9.5 Location and Design of U-Turn Median Openings (pages 9-165 and 9-166)

Proposed Revision to Green Book

~~Wide medians are uncommon in highly developed areas. Consequently,~~ Special U-turn designs, called loons, should be considered where right-of-way is restricted. ~~speeds are low, and signal control is used downstream to provide sufficient gaps in the traffic stream~~



Source: Michigan Department of Transportation ***Geometric Design Guide 670***

Figure A-18. Example graphic for new Green Book Figure 9-XJ. Typical loon design to facilitate U-turning traffic on arterials with restricted median widths.

In conditions where the U-turn crossover is unsignalized, provisions for sufficient gaps in the traffic stream through the use of an upstream signal or due to natural gaps appearing in the traffic stream due to low volumes may be needed. Further, when establishing the clearance intervals for the signalized crossover, it is essential to provide additional time to account for the extra travel distance required for drivers to navigate the loon. Median widths of 2.4 to 12.5 m [~~7~~ 8 to ~~40~~ 41 ft] may be used for U-turn openings to permit passenger vehicles or single-unit trucks to turn from the inner lane in one direction onto the shoulder of a four-lane divided highway in the other direction.

This special U-turn feature can be incorporated into the design of an urban roadway section by constructing a short segment of shoulder area along the outside edge of the traveled way across from the U-turn opening (see Figure 9-XJ). The outside curb-and-gutter section would then be carried behind the shoulder area and the shoulder would be designed as a pavement. Through the use of loons, agencies can realize the safety and operational benefits of a divided roadway using median U-turns without the high cost of acquiring enough land along the entire corridor to provide sufficient median width.

Discussion

The Green Book states that “Wide medians are uncommon in highly developed areas.” This statement is not supportable, as such, it should be removed. In locations where median roadways are common, such as Michigan, Florida, Texas, and North Carolina, it is not unusual to find wide medians in highly developed areas. Telegraph Road in Detroit has a median that varies from 60 ft to 100 ft for most of its length.

The Green Book states that “Consequently, special U-turn designs should be considered where right-of-way is restricted, speeds are low, and signal control is used downstream to provide sufficient gaps in the traffic stream.” There are several issues associated with this statement:

- The common term for “special U-turn designs” is “loon.”
- The use of loons has successfully been incorporated on many rural freeways as well as urban freeways. The suggestion that loons should be considered where “speeds are low” does not support this and should be cut. The work done by Sisiopiku and Aylsworth-Bonzelet (126, 127) showed that the operation of loons was independent of urban or rural placement and signal control strategies.
- It is unclear how signal control “downstream” would create gaps in the traffic stream. As such, this should be changed to “upstream”. While this is helpful in the urban setting, in the rural setting it is also sufficient to have low enough traffic volumes for gaps to be naturally present. This should also be reflected.
- The need to provide gaps is only necessary if the crossover is unsignalized.

The Green Book states that “Median widths of 2 to 12 m [7 to 40 ft] may be used for U-turn openings to permit passenger vehicles or single-unit trucks to turn from the inner lane in one direction onto the shoulder of a four-lane divided highway in the other direction.” However, Table 9-30 (p. 9-166) shows values of 2 m and 8 ft for passenger and 12 m and 41 ft for SU-30. This does not concur with the text. It appears that the decimal portion of the values listed for the metric chart were dropped (not rounded) from the table resulting in this discrepancy. As such, it is suggested that the text be corrected to read “2.4 to 12.5 m [8 to 41 ft]” to address this discrepancy. Further, Table 9-30 Metric should be updated to include the decimal portion of the result so that the two tables agree.

The Green Book states “This special U-turn feature can be incorporated into the design of an urban roadway section by constructing a short segment of shoulder area along the outside edge of the traveled way across from the U-turn opening. The outside curb-and-gutter section would then be carried behind the shoulder area and the shoulder would be designed as a pavement.” The provision of an illustration is critical as the use of loons is not typical and may be difficult to fully understand without one.

It should be emphasized that through the use of loons, agencies can realize the safety and operational benefits of a divided roadway using median U-turns without the high cost of acquiring enough land along the entire corridor to provide sufficient median width.

Sisiopiku and Aylsworth-Bonzelet (*126, 127*) found that when establishing the clearance intervals for the signalized crossover, it is essential to provide additional time to account for the extra travel distance required for drivers to navigate the loon. This should be noted.

9.9 Indirect Left Turns and U-Turns, 9.9.5 Location and Design of U-Turn Median Openings (page 9-165)

Proposed Revision to Green Book

Where U-turn openings are proposed for access to the opposite side of a multilane divided street, they should be 15 to 30 m [50 to 100 ft] in advance of the next downstream left-turn lane. For U-turn openings designed specifically for the purpose of eliminating left-turn movement at a major intersection, they should be downstream of the intersection~~, preferably~~. In an urban setting, they should be midblock between adjacent crossroad intersections. This type of U-turn opening should be designed with a median left-turn lane for storage. In a rural setting, they should be between 300 to 450 m [1000 to 1500 ft] apart. Additionally, a U-turn opening can be provided upstream of the major intersection to remove traffic wishing to make a U-turn from that intersection.

Discussion

The Green Book states "For U-turn openings designed specifically for the purpose of eliminating left-turn movement at a major intersection, they should be downstream of the intersection, preferably midblock between adjacent crossroad intersections." This statement raises several issues:

- Locating the crossover "midblock between adjacent crossroad intersections" implies that this application is in an urban setting. This should be specified.
- Guidance should then be given for locating the crossover in a rural setting. Examples of values being used include the following:
  - Arizona (Section 1060—Median Openings): "In a rural area, the median opening is not less than 1320 feet from an intersection with an improved public road or another median opening." (*128*)
  - Florida (Section 1.3 Department Policy on Medians and Median Openings): Minimum median opening spacing for directional crossovers for access class 2 and 3 roadways is 1320 feet (400 m). (*129*)
  - Maryland (Section 10.8 Median Crossover Spacing): Identifies minimum median crossover spacing standards for state highways in rural settings of 3000 ft for primary highways (partially controlled and uncontrolled), 1500 ft for secondary highways (arterial routes) and 1000 ft for secondary highways (collector routes). (*130*)
  - Virginia (Appendix F—Access Management Design Standards for Entrances and Intersections page F-23): Identifies minimum median crossover spacing standards of 1320 ft for a rural principal arterial with legal speed limit in excess of 35 mph. (*131*)
- It should be pointed out that they can also be upstream of the intersection (e.g., Texas U-turn), which removes traffic wishing to make a U-turn from the primary intersection.

9.9 Indirect Left Turns and U-Turns, 9.9.5 Location and Design of U-Turn Median Openings (page 9-165)

Proposed Revision to Green Book

> Normally, U-turns should not be permitted from the through lanes. However, where medians have adequate width to shield a vehicle stored in the median opening, through volumes are low and left-turn/U-turns are infrequent, this type of design may be permissible. Minimum widths of median to accommodate U-turns by different design vehicles turning from the lane adjacent to the median are given in Table 9-30. These dimensions are for a four-lane divided facility. If the U-turn is made from a median left-turn/U-turn lane, the width needed is the separator width; the total median width needed would include an additional 3.6 m [12 ft] for a single median turn lane.
>
> At major intersections, many jurisdictions allow both left turns and U-turns to be made around the curbed nose at the end of a left-turn lane. Where dual left-turn lanes are needed ~~along a street with a raised-curb median~~ and the turning volume of large trucks is high, left turns and U-turns may be permitted from the inside lane and left turns only may be allowed from the outside turn lane. However, when the turning volume of large trucks is low, a dual lane crossover maneuver may be permitted allowing both lanes to make a U-turn movement (Figure 9-XK). Under this condition, the minimum width of the median opening is 11 m [36 ft], which does not accommodate a large truck turning adjacent to another vehicle.

Discussion

The discussion of how multiple left turns impact the median opening design does not support the state of the practice. The Green Book states: "Where dual left-turn lanes are needed along a street with a raised-curb median, left turns and U-turns may be permitted from the inside lane and left turns **only** (emphasis added) may be allowed from the outside turn lane." There are many locations that use dual U-turns through a directional median opening (e.g., Detroit, MI). The inside left-turn lane turns to the inside lane of the arterial, while the outside left-turn lane turns to the outside lane of the arterial. The width of the median opening for the crossover must be increased to 11 m [36 ft] and the design does not accommodate a large truck turning adjacent to another vehicle. Further, dual left-turn can be provided at locations that do not have a raised-curb median. Sources of information include the following: Michigan DOT *Geometric Design Guide 670* (*102*), Michigan DOT *Road Design Manual* (*103*), and FHWA *Alternative Intersections/Interchanges: Informational Report* (*104*).

The information presented in Source: Michigan Department of Transportation ***Geometric Design Guide 670.***

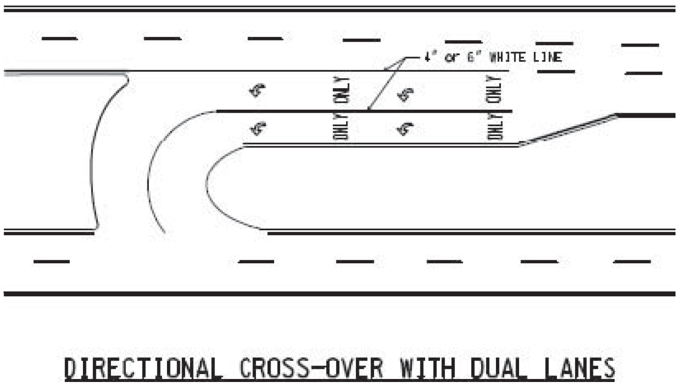
Figure A-19 and Source: Michigan Department of Transportation ***Geometric Design Guide 670.***

Figure A-20 should be combined into a single figure.

9.10 Roundabout Design (pages 9-167 to 9-169)

Proposed Revision to Green Book

> {Add this paragraph to the end of the section.}
>
> The use of roundabouts is currently evolving in the United States. As such, public outreach and education are a vital factor in realizing the improvements in traffic operations and safety that can be achieved with these designs. Additional information regarding outreach resources to help obtain public support for roundabouts is presented in the Federal Highway Administration Roundabout Outreach and Education Toolbox (*132*).



Source: Michigan Department of Transportation ***Geometric Design Guide 670***

Figure A-19. Example graphic for new Green Book Figure 9-XK. Dual U-turn directional crossover design (part A).

Discussion

The use of roundabouts is evolving in the United States. Because of this, public outreach and education are still vital factors in realizing the improvements in traffic operations and safety that can be achieved with these designs. This should be called out and resources identified, such as the FHWA publication on *Roundabout Outreach and Education Toolbox* (*132*).

9.10 Roundabout Design, 9.10.2 Fundamental Principles (page 9-173)

Proposed Revision to Green Book

{The following paragraph should be added to the end of the section entitled "Lane Balance and Continuity."}

Right-turn bypass lanes, also called slip lanes, can be implemented in conventional and innovative roundabout intersections to increase the capacity. A bypass lane is a separate right-turn lane that lies adjacent to the roundabout and allows right-turning movements to bypass the roundabout. There are three configurations for the bypass lane: slip lane without an acceleration lane stop, slip lane without an acceleration lane yield, and slip lane with free-flow entry. In areas with bicycle and pedestrian activity, bypass lanes should only be used where necessary as the entries and exits of bypass lanes can increase conflicts with pedestrians, bicyclists, and with merging on the downstream leg.

Discussion

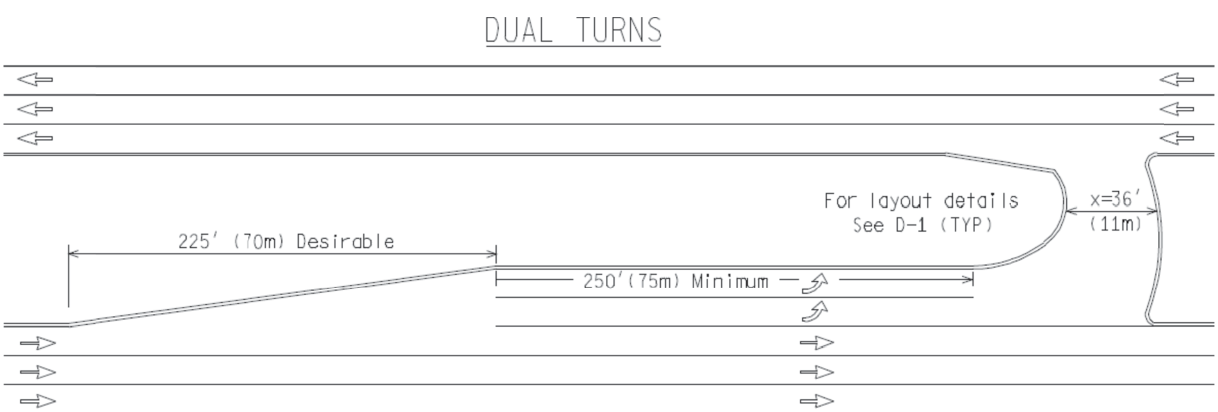
The use of a right-turn bypass is not covered in the Green Book. This would fit well at the end of the Lane Balance and Lane Continuity section. There are three configurations for the bypass lane: slip lane without an acceleration lane stop, slip lane without an acceleration lane yield, and slip lane with free-flow entry. Sources for additional information are available in the following: Mauro et al. (*133*), Al-Ghandour et al. (*134*), and NCHRP Report 672 (*135*).

9.10 Roundabout Design, 9.10.2 Fundamental Principles (page 9-174)

Proposed Revision to Green Book

{The following paragraph and figure should be added to the end of the section entitled "Appropriate Natural Path."}

The turbo-roundabout concept has emerged as a viable alternative to conventional multilane roundabouts. This design addresses functional problems of conventional multilane roundabouts by using raised splitter islands to divide traffic streams, which eliminates weaving maneuvers, forces drivers to stay in the correct lane and reduces driving speeds. Turbo-roundabouts force circulating traffic flows to use spiral pathways such that each entering lane has fixed destinations. This requires drivers to choose the correct lane for entry to



Source: Michigan Department of Transportation ***Geometric Design Guide 670***

Figure A-20. Example graphic for new Green Book Figure 9-XK. Dual U-turn directional crossover design (part B).

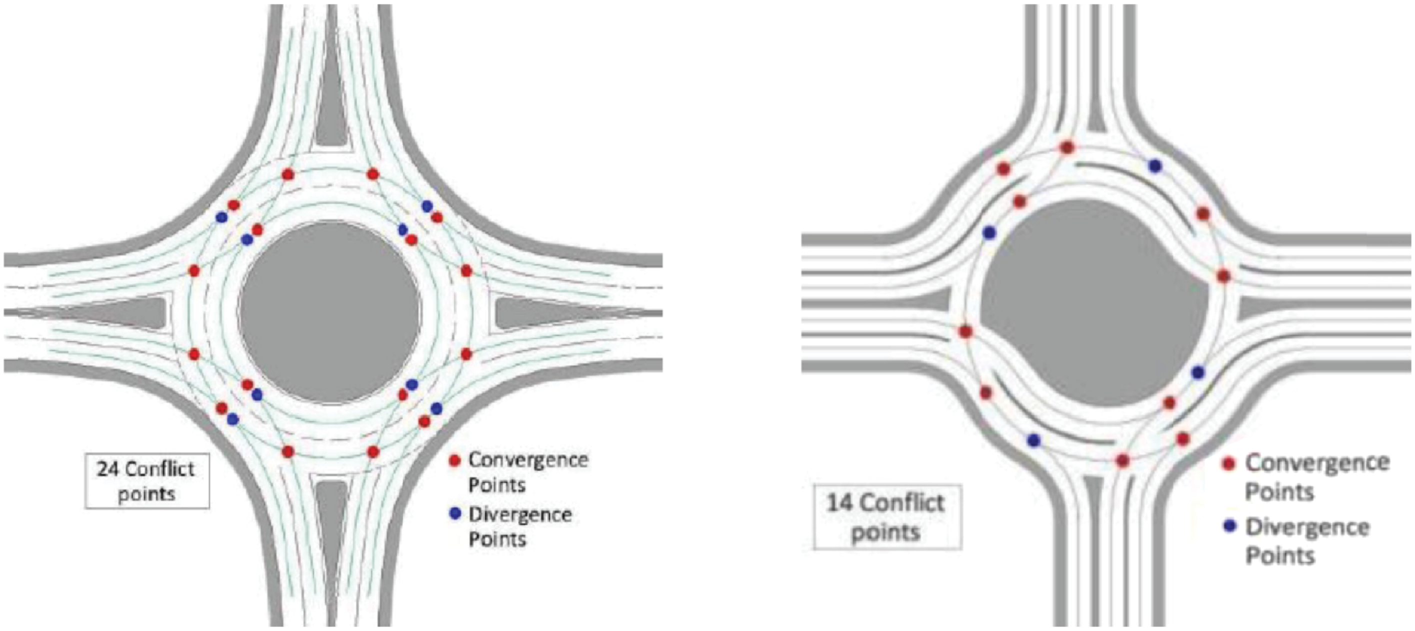


Figure 4. Conflict points in a conventional double-lane roundabout and in a turbo-roundabout
Source: Silva, Ana, Silvia Santos and Marco Gaspar, "Turbo-roundabout Use and Design," CITTA 6th Annual Conference on Planning Research.

Figure A-21. Example graphic for new Green Book Figure 9-XL. Conflict points in a conventional multilane roundabout as compared to a turbo roundabout.

the turbo roundabout based on their destination. Because of this, turbo-roundabouts do not permit U-turn maneuvers. Figure 9-XL illustrates the conflict points for a conventional multilane roundabout as compared to a turbo roundabout. The reduction in conflict points from 24 to 14 represents a global reduction in crash probability, but those conflicts associated with the turbo roundabout may result in higher severity crashes due to increased impact angle. Turbo-roundabouts have a 40 to 70% reduction in crash risk as compared to multilane roundabouts.

Discussion

Turbo roundabouts are not included. This would fit well at the end of the Appropriate Natural Path section. The physical separation between lanes helps prevent sideswipe crashes and provides positive guidance to drivers who may be unfamiliar with the operation of a roundabout. Published research shows between a 40% and 70% reduction in crash risk (*136, 137*). Fortuijn (*136*) found a 70% lower crash risk when a double-lane roundabout is converted into a turbo roundabout. Mauro et al. (*137*) reported 40–50% reduction in total accident rate based on conflict analysis techniques applied to nine layouts with different demand scenarios and 20–30% reduction in number of potential accidents with injuries. Sources for additional information are available in the following: Giuffre et al. (*138*), Giuffre et al. (*139*), Vasconcelos et al. (*140*), and Silva et al. (*141*).

9.10 Roundabout Design, 9.10.2 Fundamental Principles (page 9-175)

Proposed Revision to Green Book

{The following paragraph should be added to the end of the section entitled "Design Vehicle."}

On larger statewide facilities, it may be necessary to accommodate large WB-67 trucks or even oversized vehicles, sometimes called superloads. Special consideration for the size and tolerances of these vehicles may need to be provided in design and construction. Typical factors that may need to be considered include:

- Modification to the truck apron and central island design.
- Widened entry and exit lanes.
- Inclusion of right-turn bypass lanes.
- Use of gated pass-throughs or tapered center islands to support through movements.
- Review of lane striping.
- Installation of removable signs with setbacks for permanent fixtures (light poles).
- Identification of maximum heights for splitter islands.
- Identification of under-clearance for lowboy vehicles.
- Consideration of truck stability with regards to mountable aprons, curbing and islands.

Discussion

There is no discussion regarding accommodation of oversize/overweight vehicles. This discussion would fit well at the end of the Design Vehicle section. At a minimum, a list of things to "watch for" as a designer should be added (e.g., consider lowboy vehicle designs which could hang up, maximum height of splitter islands). Sources for additional information are available in the following: Russell et al. (*142*), Park et al. (*143*), NCHRP Report 672 (*135*).

9.10 Roundabout Design, 9.10.2 Fundamental Principles (page 9-175)

Proposed Revision to Green Book

An overarching consideration at roundabouts is the accommodation of visually impaired pedestrians. Pedestrians with vision impairments face several challenges at roundabouts. These challenges magnify the need to maintain slow vehicle speeds with the crosswalk area, to provide intuitive crosswalk alignments and to provide design elements that encourage drivers to yield to pedestrians in a predictable manner. The use of pedestrian hybrid beacons (previously known as HAWKs) and raised crosswalks have been shown to be effective (*65*).

Discussion

The information above does not provide any guidance as to which devices or design elements may encourage drivers to yield to pedestrians in a predictable manner. The discussion should be expanded to include the results of NCHRP Report 674 (*65*) as shown above.

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Abbreviations and acronyms used without definitions in TRB publications:

A4A	Airlines for America
AAAE	American Association of Airport Executives
AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ACI–NA	Airports Council International–North America
ACRP	Airport Cooperative Research Program
ADA	Americans with Disabilities Act
APTA	American Public Transportation Association
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
ATA	American Trucking Associations
CTAA	Community Transportation Association of America
CTBSSP	Commercial Truck and Bus Safety Synthesis Program
DHS	Department of Homeland Security
DOE	Department of Energy
EPA	Environmental Protection Agency
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FMCSA	Federal Motor Carrier Safety Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
HMCRP	Hazardous Materials Cooperative Research Program
IEEE	Institute of Electrical and Electronics Engineers
ISTEA	Intermodal Surface Transportation Efficiency Act of 1991
ITE	Institute of Transportation Engineers
MAP-21	Moving Ahead for Progress in the 21st Century Act (2012)
NASA	National Aeronautics and Space Administration
NASAO	National Association of State Aviation Officials
NCFRP	National Cooperative Freight Research Program
NCHRP	National Cooperative Highway Research Program
NHTSA	National Highway Traffic Safety Administration
NTSB	National Transportation Safety Board
PHMSA	Pipeline and Hazardous Materials Safety Administration
RITA	Research and Innovative Technology Administration
SAE	Society of Automotive Engineers
SAFETEA-LU	Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (2005)
TCRP	Transit Cooperative Research Program
TEA-21	Transportation Equity Act for the 21st Century (1998)
TRB	Transportation Research Board
TSA	Transportation Security Administration
U.S.DOT	United States Department of Transportation