Left-Turn Accommodations at Unsignalized Intersections

DETAILS

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CONTRIBUTORS

Fitzpatrick, Kay; Brewer, Marcus A.; Eisele, William L.; Levinson, Herbert S.; Gluck, Jerome S.; and Lorenz, Matthew R.

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

NCHRP REPORT 745

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Kay Fitzpatrick
Marcus A. Brewer
William L. Eisele
TEXAS A&M TRANSPORTATION INSTITUTE
College Station, TX

Herbert S. Levinson
Wallingford, CT

Jerome S. Gluck
Matthew R. Lorenz
AECOM
New York, NY

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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Kay Fitzpatrick, Senior Research Engineer, TTI, was the Principal Investigator. The other authors of this report are: Marcus A. Brewer (Assistant Research Engineer, TTI), Herbert S. Levinson (Principal, Herbert S. Levinson, Transportation Consultant), Jerome Gluck (Associate Vice President, AECOM), Matthew R. Lorenz (Senior Traffic Engineer, AECOM), and William L. Eisele (Research Engineer, TTI). The work was performed under the general supervision of Dr. Fitzpatrick.

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FOREWORD

By B. Ray Derr
Staff Officer
Transportation Research Board

This report presents guidance for the selection and design of left-turn accommodations at unsignalized intersections. Eleven case studies of typical situations illustrate the use of the guidance. The report will be useful to geometric designers and traffic engineers that deal with unsignalized intersections.

Maximizing the potential capacity of existing roadways is a priority in light of growing traffic demands and the diminishing resources to develop more capacity. Left turns at unsignalized intersections, including driveways, cause delay and may reduce safety. The decision to install a left-turn accommodation is a complex one as state and local transportation agencies weigh the left-turn demand, the cost of the accommodation, and the anticipated operational and safety benefits. Clear and consistent application of left-turn accommodations is important for mitigating the impacts of left turns, both for reconstruction projects and for the permitting of new access points.

In NCHRP Project 03-91, the Texas A&M Transportation Institute and their research team interviewed state and local elected officials and transportation agency administrators, business owners, and developers to determine their concerns related to providing left-turn accommodations at unsignalized intersections and identify performance measures that may influence these decisions. They then developed a process for determining whether a left-turn accommodation is justified at an unsignalized intersection and, if so, the types of accommodations that are appropriate. The process considers safety, operational efficiency, and construction costs. Design guidance was then developed for typical left-turn accommodations. The likely benefits and impacts of accommodations are described.

This report includes eleven design examples illustrating use of the guidance. These examples are based on actual locations where left-turn treatments were considered, evaluated, and/or installed. Two of the examples involve analysis of a proposed development.

The contractor's final report providing background information for the project is available on the TRB website. In addition to describing the work that was done, Appendix F presents a legal review of the impact of essential nexus and rough proportionality on development considerations.

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

Left-Turn Accommodations at Unsignalized Intersections

NCHRP Project 3-91, "Left-Turn Accommodations at Unsignalized Intersections," had two primary research goals:

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

The second of those goals led to the decision to create this design guide to facilitate the distribution of findings from that project and to assist practitioners in their efforts when installing, or deciding when to install, left-turn lanes at unsignalized intersections.

Left-turn movements at unsignalized intersections and driveways, especially those that are made from lanes that are shared with through traffic, cause delays and adversely impact safety. The warrants used by many jurisdictions for when to provide left-turn lanes are based on volume thresholds and a queuing model introduced in research from the mid-1960s. Recent research has indicted that many of the assumptions used in those warrants are dated and need to be reexamined.

The left-turn lane warrants developed from NCHRP Project 3-91 include consideration of the following:

- Rural or urban/suburban environment,
- Number of lanes on the major roadway,
- Number of approaches to the intersection,
- Peak hour left-turn lane volume, and
- Major roadway volume.

This design guide was developed with a focus on practitioner use. It does not contain a comprehensive description of NCHRP Project 3-91 or how the conclusions were developed. Rather, it focuses on designs and traffic control treatments for left-turn lanes. Details of the research project are documented in the final research report, which is separate from this design guide.

The design guide discusses the basic information (e.g., traffic volumes, speeds, crash history, and delay or gap acceptance data) a practitioner needs to make a decision on whether to install a left-turn lane and how the practitioner can compile that information. It also sets forth a process by which the practitioner can establish and apply design criteria, both in the decision to install a left-turn lane and in the process of determining the specific design elements for that lane.

Warrants for installing left-turn lanes are provided in tabular and graphical form for rural two-lane roadways, rural four-lane roadways, and urban/suburban arterials. Warrants for bypass lanes on rural two-lane roadways also are included. The volume thresholds for these warrants are

typically lower than those described in various states' manuals and other guidance documents, with installations warranted for as few as five left-turning vehicles in the peak hour, depending on the opposing through volume. These warrants are based on a benefit-cost analysis conducted in NCHRP Project 3-91.

The design guide also contains recommendations for appropriate dimensions for specific design elements, such as:

- Lane width (as wide as the adjacent through lane, but at least 10 ft),
- Taper length (8:1 [L:T] for design speeds up to 30 mph and 15:1 [L:T] for design speeds of 50 mph) and type (bay taper or approach taper; "shadowed" or direct entry),
- Deceleration length (preference for no deceleration in the through lane, though 10 mph may be allowed), and
- Storage length (enough to store expected number of design vehicles during a critical period, but at least 40–50 ft, or two passenger car lengths).

A discussion of intersection sight distance, median design, and channelization and offset are included, as well as general principles for bypass lane design and pedestrian accommodation.

Traffic control devices are important complements to the geometric design of unsignalized intersections. The guide discusses commonly used signs, markings, and illumination, based on guidance from the MUTCD and lighting guides from AASHTO, ITE, and IESNA. The appropriate types (e.g., regulatory, warning, guide), sizes, and placement of signs are described, as are application guidelines for both signs and markings. Commonly used principles related to type, intensity, and location of lighting are explained, and provisions for accommodating future signalization are also included.

The guide also contains a series of 11 design example case studies, based on actual intersections where left-turn treatments were evaluated and installed. These case studies include installations of exclusive left-turn lanes in developing areas, a left-turn lane as part of a road diet program, a J-turn intersection, a bypass lane (or "passing blister"), and a signalized left-turn lane. Design examples provide the practitioner with "real-world" scenarios that are similar to those the practitioner would actually encounter, along with possible solutions and the methods by which those solutions can be evaluated and installed.

CHAPTER 1

Introduction

Background of Study

Left-turn movements at unsignalized intersections, including driveways—especially movements that are made from lanes that are shared with through traffic—cause delays and adversely impact safety. The warrants used by many jurisdictions for when to provide left-turn lanes are based on volume thresholds and a queuing model introduced in research from the mid-1960s. Recent research has indicted that many of the assumptions used in those warrants are dated and need to be reexamined. Part of the concerns regarding the existing warrants were other factors that should be considered when researching left-turn lane warrants, such as the cost element, particularly when right-of-way issues are involved, and the rational nexus, when the left-turn accommodation may be related to a proposed new development.

These and related issues led to the creation of NCHRP Project 3-91, "Left-Turn Accommodations at Unsignalized Intersections." The project had two primary research goals:

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

The second of these goals led to the decision to create this design guide to facilitate the distribution of findings from that project and to assist practitioners in their efforts when installing, or deciding when to install, left-turn lanes at unsignalized intersections. Documentation of how the left-turn lane warrants were developed is contained in the final research report from the project (*1*).

The left-turn lane warrants developed from NCHRP Project 3-91 include consideration of the following:

- Rural or urban/suburban environment,
- Number of lanes on the major roadway,
- Number of approaches to the intersection,
- Peak-hour left-turn lane volume, and
- Major roadway volume.

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

- Sight distance relative to the position of the driver and
- 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, etc.).

The practitioner must consider a number of issues when making decisions about planning and designing the lane. Some of these issues include:

- Design vehicle,
- Width of the turn lane,
- Need for an island of appropriate size for pedestrian refuge,
- Pedestrian facilities,
- Length for deceleration,
- Taper/transition length,
- Length storage for turning vehicles,
- Signage,
- Pavement markings,
- Illumination,
- Position of driveways/intersections in the vicinity of the left-turn lane, and

- Potential of signalizing the intersection in the foreseeable future.

Proper consideration of these issues, and the ability to make informed decisions about them, will improve the likelihood that a left-turn lane is installed at an appropriate location and that its design will lead to improved operations during its service life.

Purpose and Scope of Guidelines

The design guide was developed with a focus on practitioner use. It does not contain a comprehensive description of the research project or how the conclusions were developed. Rather, it focuses on designs and traffic control treatments for left-turn lanes. Details about the research project are documented elsewhere (*1*).

Organization of Design Guide

This design guide has five chapters, including this introductory chapter. Chapter 2 discusses key steps in the planning and design process and how to compile and use the necessary information to complete those steps. It also contains the warrants the practitioner should consider when deciding whether to install a left-turn lane at a particular location. Chapter 3 provides a summary of key geometric design criteria and considerations specific to left-turn lanes at unsignalized intersections. Chapter 4 discusses appropriate traffic control devices and illumination, including provisions for future signalization. Chapter 5 contains a variety of case studies. The practitioner can refer to these case studies for examples of prior left-turn lane installations, which illustrate decisions and considerations made by other practitioners in similar situations.

CHAPTER 2

Planning and Design Process

Introduction

Before installing a left-turn lane (or any other roadway improvement), it is necessary to consider the characteristics of the location where it would be installed. These characteristics guide the practitioner's decisions about whether to install the lane and what specific design criteria need to be emphasized to optimize the operation of the lane at that location. This chapter sets forth guidelines for establishing and applying decision criteria for providing left-turn lanes at unsignalized intersections. These guidelines can be useful to transportation agencies in planning new roadways and upgrading existing facilities.

The information in this chapter is closely related to that presented in Chapter 3 because much of the information necessary to make the fundamental decision to install a left-turn lane is also used in the subsequent decisions about the details of the design of the lane. Therefore, the practitioner should consider the material in that chapter in conjunction with the following sections when making decisions early in the process of planning and designing a left-turn lane installation.

Assemble Basic Information

There are some common reasons that practitioners consider the installation of a left-turn lane to improve an intersection.

- Speeds are too high to safely make left turns to or from a particular roadway.
- There is a trend or pattern of crashes involving left-turning vehicles, or rear-end or sideswipe/weaving crashes as through vehicles interact with queued vehicles.
- Drivers have to wait a long time to make a left turn.
- There are a high number of left-turning vehicles.

The thresholds that practitioners apply to these situations (e.g., what speed is "too high") can vary depending on local or state guidelines, previous experience with left-turn lanes in that area, and input from local stakeholders. In addition, the perception of these issues (whether real or imagined) also can initiate a review of an intersection, particularly when it comes to speed and/or crashes. As a result, basic information needed to assess the validity of those issues is commonly collected at the start of the planning process. In addition to the items discussed below, information also is needed regarding other conditions in the area, such as how left turns are treated at other locations along the corridor in question and the spacing between a location under study and upstream and downstream traffic signals.

The basic information needed for use with the developed left-turn lane warrants includes:

- Development (rural or urban/suburban),
- Number of lanes on the major roadway (two or four),
- Number of approaches (three legs or four legs),
- Peak-hour left-turn lane volume (left-turn vehicles per hour), and
- Major roadway volume (vehicles per hour per lane).

Other information traditionally used in traffic engineering studies includes:

- Volume,
- Speed,
- Crashes, and
- For selected locations delay and/or gap acceptance.

Basic Information

The basic geometry of the intersection needed for use with the warrants is the number of lanes on the major roadway and the number of approaches to the intersection. The number of approaches and the development type (rural or urban/suburban) are included in the warrants because the crash prediction methodology used to develop the warrants varied by these features. Rural crash prediction equations vary by number of lanes on the major roadway, so the warrants for rural highways also vary by number of lanes.

Volume

The peak-hour left-turn volume and major road volume are needed for use in the left-turn warrants.

Quantifying the overall traffic volumes at a candidate intersection can provide a better understanding of the conditions at a site. An aggregate volume count can be conducted with a speed study if automated traffic counters are used. The two studies can then cover the same amount of time and be reviewed for patterns over time of day or day of the week.

However, it is also important that these volumes are identified not only by time of day, but by turning movement. A comprehensive turning-movement count quantifies the number of vehicles on each leg of the intersection that turned left, turned right, or proceeded straight through the intersection. These turning-movement counts must be done manually or be collected by video and later manually reviewed. While this type of count is more resource intensive, it provides the information needed to know what the left-turn demand is on each approach, and it also identifies the opposing traffic volumes through which turning drivers must complete their turning maneuvers. When analyzing the existing counts, it is important to consider growth in future left-turn demand associated with plans for future development.

If the lane is being considered because of a particular traffic generator (e.g., a sporting or concert venue or a seasonal event), then the volume data should be collected when that generator is expected to produce the traffic being considered.

Speed

A speed study to determine the prevailing speeds at a particular intersection is a straightforward way to evaluate the effect of speeds on turning movements. Commonly, these speeds are collected through the use of automated traffic counters, usually deployed at selected spot locations on each approach (and perhaps departure) leg of the intersection. Alternatively, radar or laser speed devices can be used to collect a sample of spot speeds for a particular period of time, such as the peak hour.

Detailed procedures on conducting spot-speed studies can be found in other sources (*2*). If concerns exist about changing conditions, speed data can be collected on several days or through the course of a week to identify speed patterns over time. An entire week allows the practitioner to see trends on weekdays versus weekends, day versus night, and other time-sensitive comparisons that could influence the decisions related to the left-turn lane.

The result of such a study is a spreadsheet of data showing basic speed statistics (e.g., average, minimum, maximum, 85th percentile) for the entire study period as well as other divisions of time (e.g., 24-hour periods, 6-hour periods, 1-hour periods for peak times). These statistics help illustrate patterns and trends in operating speeds at the intersection, which can be used to make informed decisions about the design of the turning lane.

Crash History

If an intersection has a problem (or a perceived problem) with left-turn-related crashes, then a crash study is necessary to determine the extent of the problem. A review of recent crash reports can provide insight into the nature of the problem and how a left-turn lane may be a suitable countermeasure. For example, if crashes are occurring predominantly on one leg of the intersection, then it may be necessary to install a left-turn lane only on that leg, rather than on multiple legs. Left-turn-related crashes may include rear-end crashes at or near the intersection or driveway as well as sideswipe crashes, especially for multilane streets.

Often a crash study considers the previous 12 months, though a 24- or 36-month study may be useful, particularly if the practitioner desires to estimate the effects of adjacent development or other recent changes at that location. For example, if crashes increased after the completion of a new housing development, then the intersection may need a left-turn lane, a need that did not exist prior to its completion.

Depending on the jurisdiction or the classification of the roadway, there may be an electronic database of crash data that can be searched for information. Use of the electronic database can be helpful in searching through a large number of crashes in a short period of time; however, electronic records typically do not contain the level of detail provided by printed copies of the law enforcement officers' original reports. The full report shows a diagram of the intersection that includes the movements of vehicles and locations of key objects. The report also contains a narrative of the sequence of events leading to the crash, as the officer states it in his or her own words. The details provided by the diagram and the narrative are extremely valuable in identifying patterns related to crashes and crash history at a given intersection.

Delay

A study of the delay and/or gap acceptance at an intersection can provide useful information to determine if there is a potential problem that a left-turn lane can address. If left-turning drivers must wait a lengthy amount of time to complete their turns, it can lead to further delay for the drivers waiting in the queue behind them, whether they intend to turn or travel straight through the intersection. A left-turn lane would provide storage for turning vehicles and remove impediments to through vehicles.

A delay study looks at the time it takes vehicles to travel through the intersection under prevailing conditions and compares it to the amount of time it would take the same vehicles to travel through the intersection under free-flow conditions. A discussion of delay and gap acceptance studies for two-way and all-way stop-controlled intersections, as well as the recommended procedure and equations to use, can be found in the *Highway Capacity Manual* (3).

It is important to review delay in the context of crashes. Sites with a left-turn crash problem may have high delay, but sites without a crash problem may also have delay because increasing delay can be a precursor to an increasing crash rate. As drivers wait longer to turn, they can become impatient and attempt turns when it is not actually safe to do so.

Establish and Apply Decision Criteria

While the practitioner is still early in the planning and design process, the criteria for making decisions about left-turn lane installation need to be defined. In other words, what factors are important for determining the characteristics that the lane needs to have? Some of these factors depend on the issues identified in the previous section; whether some combination of these factors is an influence on the site will partially determine what other factors need to be considered when planning and designing the new lane.

In addition to the basic contributors, other factors might be:

- Access to adjacent development, in conjunction with access management considerations and requirements;
- Available right-of-way;
- Existing roadway width;
- Safety or crash history;
- Consistency with nearby intersections;
- Sight distance restrictions;
- Speed differential concerns;
- Pedestrian traffic;
- Existing or proposed medians; and
- Available budget.

Decisions relating to left-turn installation, particularly for a left-turn lane into a new development, may be governed by access management provisions of the agencies with jurisdiction. In order to preserve mobility and provide safety for the traveling public, many transportation agencies have established regulations and programs to manage access to their roadway network. For access to a new development, state and local agencies typically use access permitting to apply access management standards to guide decisions regarding where and what access would be allowed as well as any restrictions to this access. In general, the regulations are more restrictive for major arterials, the roadways intended to accommodate higher volumes and speeds.

These factors must then be considered in conjunction with the corresponding geometric design criteria and applicable policies on planning and access management to make appropriate decisions about specific characteristics of that particular left-turn lane. For example, on an intersection approach under consideration for a left-turn lane, there is a driveway upstream of the intersection where left turns also commonly occur. Vehicles turning into that driveway may also queue in the same through lane and cause confusion both to through drivers and to drivers intending to turn left at the intersection. In order to improve operations for through vehicles and reduce the likelihood of delay or rear-end crashes as through vehicles approach queued vehicles turning into the driveway, it may be desirable or preferable to provide a left-turn lane at this location as well. The spacing between the intersection and driveway will be a major consideration in establishing the appropriate treatment. An option may be to extend the left-turn lane upstream from the intersection to the driveway to provide storage for all left-turning vehicles. However, it may also be prudent to completely restrict left-turn access to that driveway through the use of a raised median, where possible, in order to reduce confusion among left-turning drivers; prohibiting turns into the driveway would prevent the formation of two intermingled queues (one for the driveway and one for the intersection) within the same lane, and it would eliminate the subsequent potential for "jockeying for position" within the left-turn lane and the adjacent through lane as drivers turning at the intersection try to avoid the queue for the driveway.

The practitioner should identify and list all such influencing factors and document them as decisions are made throughout the planning and design phases of the installation. The decision flowchart shown in Figure 1 is an example

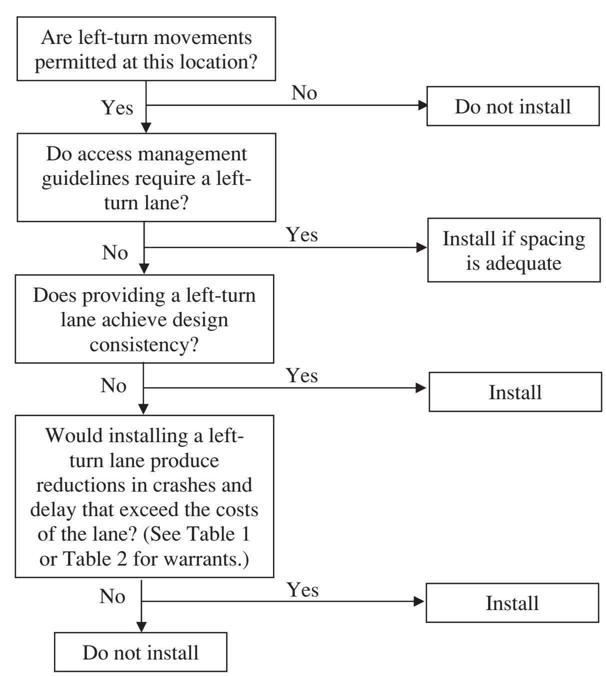


Figure 1. Example decision flowchart for installation of left-turn lane.

of the steps a designer could take to determine whether a left-turn lane is appropriate for a particular location. Where there are no applicable access management guidelines, adequate spacing and design consistency are both essential requirements to consider.

Apply Left-Turn Lane Warrants

Warrants

After compiling all of the relevant information pertaining to a particular intersection, it is necessary to determine whether that information indicates that a left-turn lane is indeed necessary or beneficial. Left-turn lanes can reduce the potential for collisions and improve capacity by removing stopped vehicles from the main travel lane. The recommended left-turn lane warrants developed based on the NCHRP Project 3-91 research (*1*) are:

- Rural, two-lane highways (see Table 1),
- Rural, four-lane highways (see Table 2), and
- Urban and suburban roadways (see Table 3).

Table 1 also present warrants for a bypass lane treatment on two-lane rural highways. Given a peak-hour left-turn volume and a particular intersection configuration (i.e., number of legs, number of lanes on the major highway), the tables show the minimum peak-hour volume on the major highway that warrants a left-turn lane or bypass lane. Figure 2 displays the warrants for rural two-lane highways graphically. Figure 3 shows graphical warrants for four-lane rural highways, and Figure 4 shows the recommended warrants for urban and suburban arterials.

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 and
- 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, etc.).

Source of Warrants—Benefit-Cost Approach

A benefit-cost approach was conducted as part of NCHRP Project 3-91 (*1*) to determine when a left-turn lane would be justified. Economic analysis can provide a useful method for combining traffic operations and safety benefits of left-turn lanes to identify situations in which left-turn lanes are and are not justified economically. The development steps included:

- Simulation to determine delay savings from installing a left-turn lane,
- Crash costs,
- Crash reduction savings determined from safety performance functions available in the AASHTO *Highway Safety Manual* (Chapter 10 discusses rural two-lane, two-way roads; Chapter 11 discusses rural multilane highways; and Chapter 12 discusses urban and suburban arterials) (*4*),

Table 1. Recommended left-turn treatment warrants for rural two-lane highways.

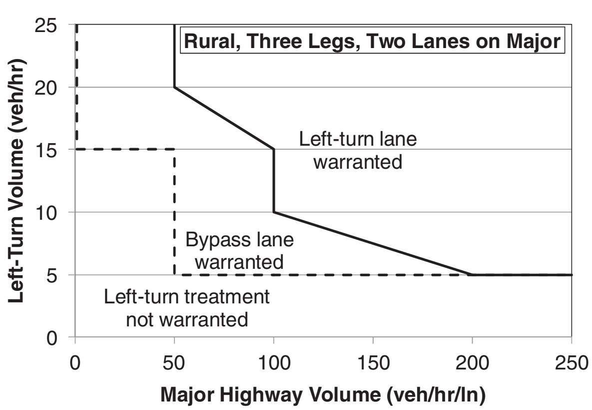
Left-Turn Lane Peak-Hour Volume (veh/hr)	Three-Leg Intersection, Major Two-Lane Highway Peak-Hour Volume (veh/hr/ln) That Warrants a Bypass Lane	Three-Leg Intersection, Major Two-Lane Highway Peak-Hour Volume (veh/hr/ln) That Warrants a Left-Turn Lane	Four-Leg Intersection, Major Two-Lane Highway Peak-Hour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

Table 2. Recommended left-turn lane warrants for rural four-lane highways.

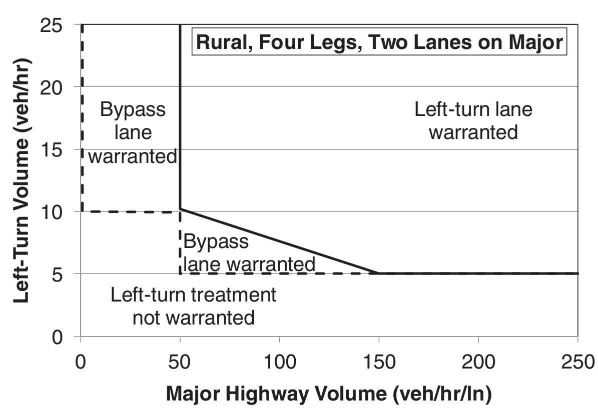
Left-Turn Lane Peak-Hour Volume (veh/hr)	Three-Leg Intersection, Major Four-Lane Highway Peak-Hour Volume (veh/hr/ln) That Warrants a Left-Turn Lane	Four-Leg Intersection, Major Four-Lane Highway Peak-Hour Volume (veh/hr/ln) That Warrants a Left-Turn Lane
5	75	50
10	75	25
15	50	25
20	50	25
25	50	< 25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

Table 3. Recommended left-turn lane warrants 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 2. Recommended left-turn treatment warrants for intersections on rural two-lane highways.

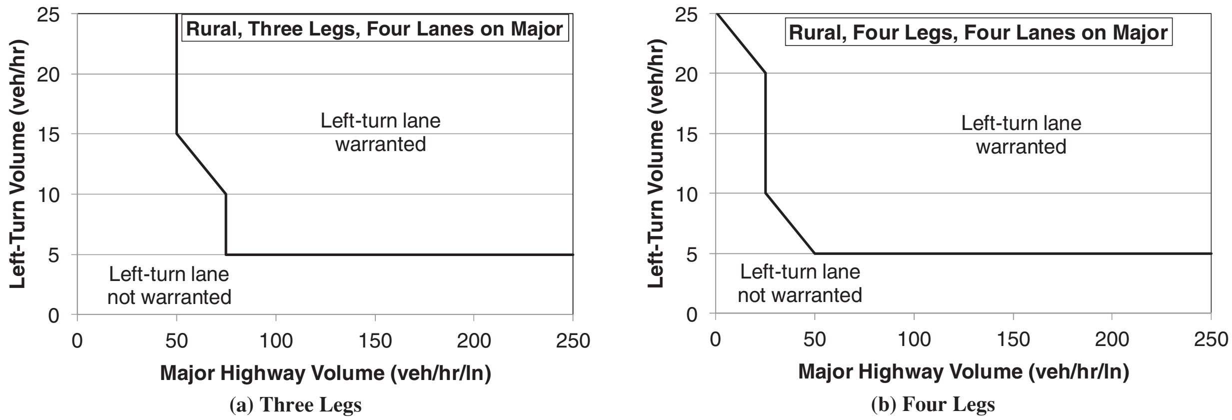


Figure 3. Recommended left-turn lane warrants for intersections on rural four-lane highways.

- Crash modification factors available in the AASHTO *Highway Safety Manual* (*4*), and
- Construction costs.

For rural conditions, different safety performance functions are provided for two- and four-lane highways and for three- and four-leg intersections. For urban and suburban arterials, prediction equations are provided for three-leg and four-leg intersections. Separate urban and suburban prediction equations are not provided based on the number of lanes on the major road approach. The prediction equations are not a function of speed limit; therefore, the developed warrants also are not a function of speed limit.

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

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

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

Left-turn lanes can reduce the potential for collisions and improve capacity by removing stopped vehicles from the main travel lane. Left-turn lane warrants were developed as part of NCHRP Project 3-91 using an economic analysis procedure for rural, two-lane highways; rural, four-lane highways; and urban and suburban roadways. The methodology presented in the NCHRP Project 3-91 report (*1*) could also be used if a transportation agency has available local values for delay

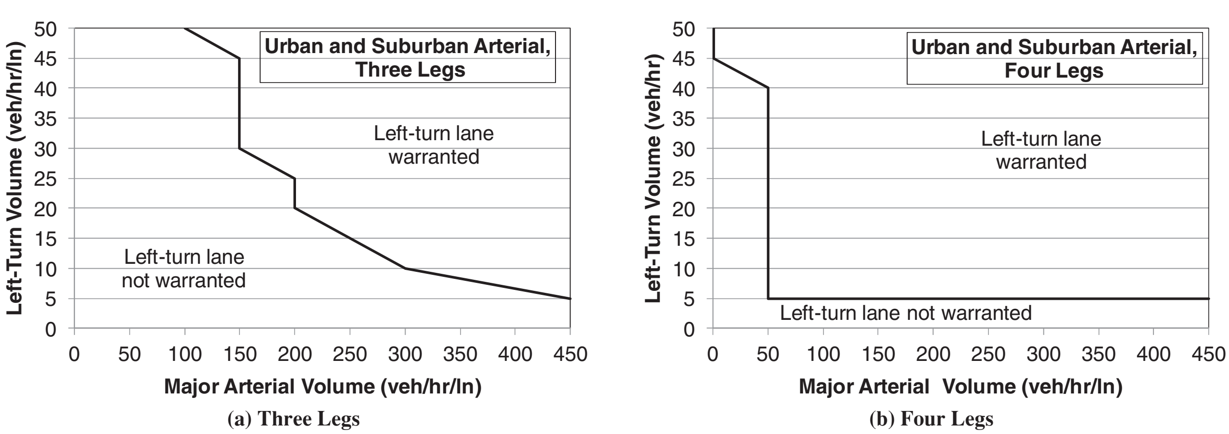


Figure 4. Recommended left-turn lane warrants for intersections on urban and suburban arterials.

reductions due to the installation of a left-turn lane, crash frequency or crash predictions, crash reduction factors, crash costs, and/or construction costs. If crash and/or delay data are available for a specific location, the benefit-cost method as described in the research report can be used to evaluate the potential benefit of installing a left-turn lane at a specific location. The available crash data should be combined with the crash predictions for the site using an empirical Bayes (EB) approach. Both the crash prediction and the EB procedures are discussed in the *Highway Safety Manual* (*4*). The EB technique is properly exercised by statisticians who have familiarity with this method and interpretation of its results. Highway agencies that desire to use this method but do not have personnel with relevant EB experience should consider employing the resources of a consultant who is experienced in the use of the method.

Prepare Designs

Once the decision to install the left-turn lane has been finalized, and the planning process has been completed—considering all of the important contributing factors in the placement of the left-turn lane—designs for the specific dimensions of the lane must be prepared. Depending on the characteristics of the intersection, it may be appropriate to prepare more than one design option and compare their relative strengths and weaknesses. Alternatively, individual design elements can be discussed and evaluated as part of an overall design plan. Either way, the elements comprising the design need to be created according to accepted geometric design principles that account for factors such as design speed and design vehicle, sight distance, storage area, deceleration area, grade, and channelization. These principles and others are discussed in Chapter 3.

CHAPTER 3

Geometric Design

Introduction

Once the decision has been made to provide a left-turn lane at a particular intersection, and the preliminary planning has been completed to determine the basic elements of the lane relative to the characteristics of the location (e.g., right-of-way, traffic volumes, etc.), the dimensions and other physical characteristics of the lane must be specified based on geometric design principles. Figure 5 shows a typical left-turn lane layout.

Selection of Design Speed

Speed is one of the most important factors considered by travelers in selecting alternative routes or transportation modes. According to the AASHTO *Green Book* (*5*), design speed is a selected speed used to determine the various geometric design features of the roadway. The selected design speed should be a logical one with respect to the topography, anticipated operating speed, adjacent land use, and functional classification of the highway. In selection of design speed, every effort should be made to attain a desired combination of safety, mobility, and efficiency within the constraints of environmental quality, economics, aesthetics, and social or political impacts. Once the design speed is selected, all of the pertinent highway features should be related to it to obtain a balanced design.

As one of the geometric design features of a particular intersection, a left-turn lane should be designed in relation to the main lanes of the adjacent roadway. The selected design speed affects the design of the subsequent elements of the turning lane, and the designer should keep in mind the interaction of vehicles between the main lanes and the turning lane when choosing the design speed. The selected design speed should be consistent with the speeds that drivers are likely to expect on a given highway facility. Therefore, the design speed for a left-turn lane should generally be equal to that of the adjacent roadway, absent constraints imposed by right-of-way or other limiting factors.

A single design speed for deceleration lanes may be appropriate for rural/undeveloped locations where speed is essentially constant throughout the day. However, in suburban/urban (developed) areas, speeds in peak periods are commonly lower than in off-peak periods, and left-turn queue lengths are typically longer in peak periods. Therefore, it is prudent to consider two speeds (and two corresponding queue storage lengths), one for peak periods and one for off-peak periods. Additionally, conditions in the morning peak period may be different than in the afternoon, most commonly for commuter routes with a high directional split that reverses from morning to afternoon. When determining the appropriate value for design speed, the value chosen should be that for the conditions that result in the longest required distance for deceleration plus queue storage.

When designing other geometric elements based on the design speed, design values that are more generous than the minimum should be used where practical, particularly on high-speed facilities. Some design features, such as taper length and sight distance, are directly related to, and vary appreciably with, design speed. Other features, such as widths of lanes and shoulders, are not directly related to design speed, but they do affect vehicle speeds. Therefore, wider lanes and shoulders should be considered for higher design speeds. Thus, when a change is made in design speed, many elements of the turning lane design change accordingly.

Selection of Design Vehicle

Key controls in geometric highway design are the physical characteristics and the proportions of vehicles of various sizes using the roadway. Different vehicles have different needs that affect their ability to navigate a left-turn lane. Therefore, it is necessary to select a vehicle, called a design vehicle, with weight, dimensions, and operating characteristics that are

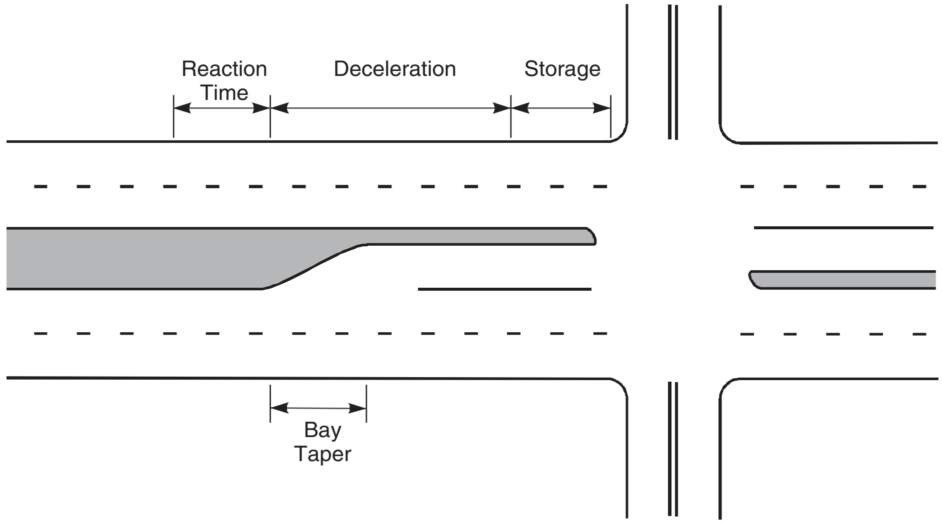


Figure 5. Typical left-turn lane layout.

representative of the vehicles likely to most commonly use the left-turn lane.

When selecting a design vehicle, the designer must first consider the mix of vehicle types that use the adjacent roadway and then which of those vehicles are expected to use the left-turn lane.

AASHTO (5) has established four classes of design vehicles:

- Passenger cars,
- Buses,
- Trucks, and
- Recreational vehicles.

Within each class, there are multiple unique design vehicles to consider. The AASHTO *Green Book* (5) provides guidance in selecting the most appropriate design vehicle, including representative turning radii and other characteristics.

The TRB *Access Management Manual* (6) provides suggested storage length per vehicle for different percent truck levels (reproduced in Table 4).

Desirable and Minimum Lane Widths

Left-turn lanes at intersections and interchanges often help to facilitate traffic movements. Such added lanes should be as wide as the through-traffic lanes, but not less than 10 ft. Of course, the width of the turning lane is dependent on the width of the median where it is located. 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 (*7*). In urban situations with constrained space and low speed, 9-ft lanes have been used, but these are the exceptions rather than the rule.

Table 4. Queue storage length per vehicle (*6*).

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

Median widths of 20 ft or more are desirable at intersections with single median lanes, but widths of 16 to 18 ft permit reasonably adequate arrangements. Where two median lanes are used, a median width of at least 28 ft is desirable to permit the installation of two 12-ft lanes and a 4-ft separator. Although not equal in width to a normal traveled lane, a 10-ft lane with a 2-ft curbed separator or with traffic buttons or paint lines, or both, separating the median lane from the opposing through lane may be acceptable where speeds are low and the intersection is controlled by traffic signals. According to draft guidelines for accessible public rights-of-way published by the United States Access Board (*8*), a median width of at least 6 ft must be provided for storage where pedestrians will be present.

Tapers

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 6 shows a left-turn lane added within a median.
- Figure 7 shows a left-turn lane that was added to an undivided two-lane highway where the through lane on the

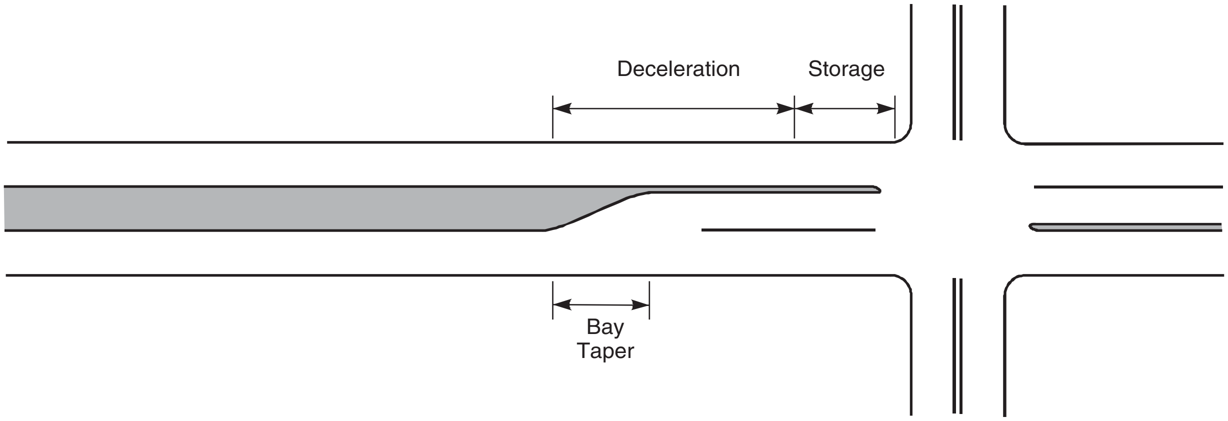


Figure 6. Left-turn lane within a median.

same approach as the added turn lane was shifted to the right the full width of the turn lane. This condition is known as a full-shadowed left-turn lane.

- Figure 8 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 (*9*).
- Figure 9 shows the condition when a lane is added to the outside edge of the approach, allowing through vehicles to pass left-turning vehicles on the right. This condition is also known as a bypass lane. The bypass lane minimizes delay to following through vehicles by allowing the vehicle following to pass the left-turning vehicle on the right, and then merge back into the through lane. Some states do not allow informal passing on the right or driving on the shoulder; therefore, the additional width for through vehicles provides a legal means of passing slowed or stopped left-turning vehicles. 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 newly added outside lane is for slower-moving traffic, and the inside existing lane is for faster-moving vehicles. For the configuration shown in Figure 9, the opposite situation is present; the newly 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 bay taper rate that is between 8:1 and 15:1 (longitudinal:transverse [L:T]). 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 constrain the lateral movement of a driver desiring to enter the left-turn lanes. This problem primarily occurs on urban curbed roadways (*5*).

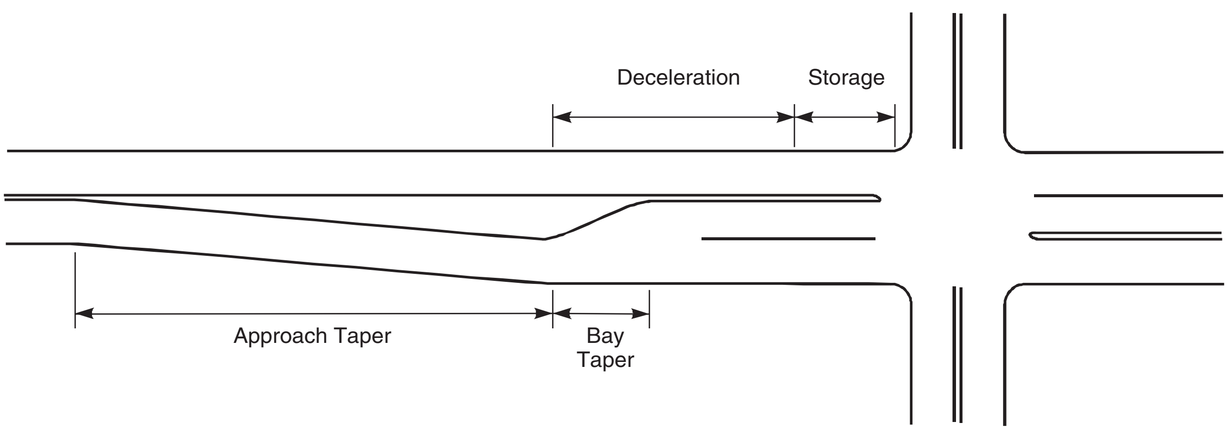


Figure 7. Full-shadowed left-turn lane.

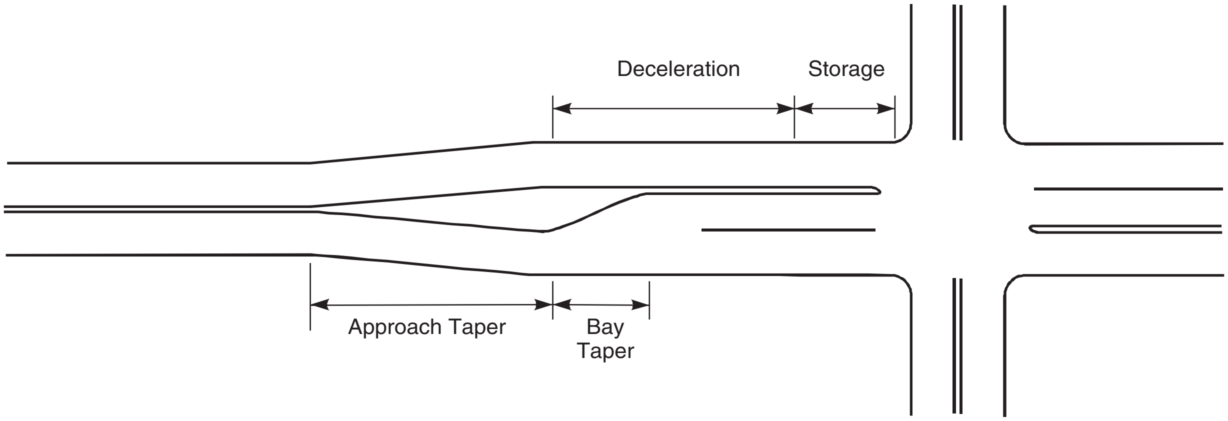


Figure 8. Partially shadowed left-turn lane.

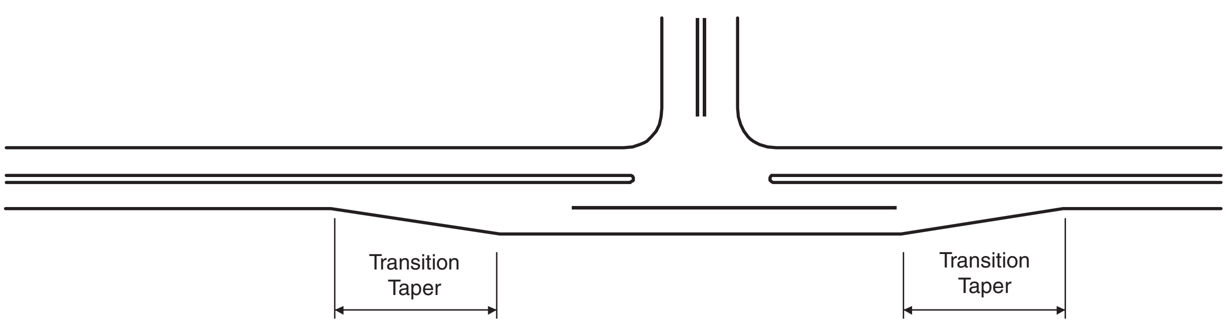
For urbanized areas, short tapers appear to produce better visual "targets" for the approaching drivers and to give more positive identification to an added left-turn lane. Short tapers are preferred for deceleration lanes at urban intersections because of slow speeds during peak periods. This results in a longer length of full-width pavement for the left-turn lane. This type of design may reduce the likelihood that entry into the left-turn lane may spill back into the through lane. Municipalities and urban counties are increasingly adopting the use of taper lengths such as 50 to 100 ft for a single-turn lane and 100 to 150 ft for a dual-turn lane for urban streets.

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 in locations 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 in such a way 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 7 and Figure 8. 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



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 9. Direct entry into left-turn lane (also known as bypass lane).

Table 5. Typical length of approach taper to add left-turn lanes.

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
Where: W = width of offset (ft) S = speed (mph)				

T-intersections, where there is no median in which to install a shadowed left-turn lane, and/or at locations 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 5. Comparisons for various speeds and offsets are shown in Table 5.

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 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 Length

Provision for deceleration clear of the through traffic lanes is a desirable objective on arterial roads and streets and should be incorporated into design whenever practical. The approximate total lengths needed for a comfortable deceleration to a stop from the full design speed of the highway are shown in Table 6. These approximate lengths are based on grades of less than 3 percent.

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. Shorter left-turn lane lengths increase the speed differential between turning vehicles and through traffic. A 10-mph differential is commonly considered acceptable on arterial roadways. Higher-speed differentials may be acceptable on collector roadways due to higher levels of driver tolerance for vehicles leaving or entering the roadway due to slow speeds

Table 6. Deceleration lengths for left-turn lanes.

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	
	AASHTO *Green Book* (5), page 714	TRB *Access Management Manual* (6), page 172	Florida Department of Transportation *2006 FDOT Design Standards* (10)	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
Blank cells = deceleration length not provided in reference document for the given design speed					

or high volumes. Therefore, the no-speed-reduction lengths given in Table 6 should be accepted as a desirable goal and should be provided where practical.

Vehicle Storage Length

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

According to the *Green Book* (*5*), at unsignalized intersections, the storage length—exclusive of taper—may be based on the number of turning vehicles likely to arrive in an average 2-minute period within the peak hour. Space for at least two passenger cars should be provided; with over 10 percent truck traffic, provisions should be made for at least one car and one truck. The 2-minute waiting time may need to be changed to some other interval that depends largely on the opportunities for completing the left-turn maneuver. These intervals, in turn, depend on the volume of opposing traffic, which the *Green Book* does not address. For additional information on storage length, the *Green Book* refers the reader to the *Highway Capacity Manual* (*3*). The equation presented in the TRB *Access Management Manual* (*6*) (and reproduced in Table 7) can be used to determine the design length for left-turn storage as described by the *Green Book*.

NCHRP Report 457 (*11*) developed suggested storage length values using equations identified from Harmelink's work (*12*) regarding storage length of left-turn bays at unsignalized intersections. The storage length equation is a function of movement capacity, which is dependent upon assumed critical gap and follow-up gap. Critical gap is defined by the *Highway Capacity Manual* as the minimum time interval in the major street traffic stream that allows intersection entry for one minor-street vehicle. Thus, the driver's critical gap is the minimum gap that would be acceptable. The time between the departure of one vehicle from the minor street and the departure of the next vehicle using the same major street gap, under a condition of continuous queuing on the minor street, is called the follow-up time.

NCHRP Report 457 used a smaller critical gap (4.1 sec as recommended in the *Highway Capacity Manual* compared to the 5.0 or 6.0 sec used by Harmelink for two-lane and four-lane highways, respectively), which resulted in shorter values than those generated by Harmelink. The assumptions made regarding critical gap or follow-up gap and the

Table 7. Equations used to determine storage length.

Equation in TRB *Access Management Manual*		
$L = \frac{V}{N_c} ks$ (1)		
Where: L = design length for left-turn storage (ft) V = estimated left-turn volume, vehicles per hour (veh/hr) N_c = number of cycles per hour. For the *Green Book* unsignalized procedure, this would be 30 (V/N is the average number of turning vehicles per cycle). k = factor that is the length of the longest queue (design queue length) divided by average queue length (a value of 2.0 is commonly used for major arterials, and a value of 1.5 to 1.8 might be considered for an approach on a minor street or on a collector where capacity will not be critical). For the *Green Book* procedure, this would be 1.0. s = average length per vehicle, including the space between vehicles, generally assumed to be 25 ft (adjustments for trucks and buses are available in several documents such as the TRB *Access Management Manual*)		
Equations Used in *NCHRP Report 457* Equations also used to generate values in Table 8		
$P(n>N) = \left(\frac{v}{c}\right)^{(N+1)}$	$c = \frac{V_o e^{-V_o t_c/3600}}{1 - e^{-V_o t_f/3600}}$	$N = \frac{ln[P(n>N)]}{ln[v/c]} - 1$
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) t_c = critical gap (sec) t_f = follow-up gap (sec)		

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 (*11, 13, 14*).

It is generally recognized that a storage area should adequately store the turn demand a large percentage of the time (e.g., 95 percent or more). A 0.5 percent limit was used for the major road left-turn bay lengths in *NCHRP Report 457* based on the recommendation of Harmelink. This smaller limit reflects the greater potential for severe consequences when a bay overflows on an unstopped, major road approach. The critical and follow-up gaps were assumed to equal 4.1 and 2.2 sec, respectively. When the critical gap of 5.0 and 6.25 sec determined in the NCHRP Project 3-91 field studies are used for critical gap (follow-up gap was 2.2 sec), the storage lengths shown in Table 8 are generated. A critical gap of 5.0 sec represents the 50th percentile, while the critical gap of 6.25 sec represents the 85th percentile value (which is preferred for design) for the data collected as part of the field studies in this project.

Each of the sources on storage length emphasize that the appropriate storage length is dependent on both the volume of turning traffic and the volume of opposing traffic. If volume data are not available, for urban and suburban streets with lower speeds (e.g., less than 40 mph), it is recommended that the minimum storage length be at least 50 ft to accommodate two cars; for high speed and rural locations, a minimum storage length of 100 ft is recommended. Some cities use 250-ft storage lanes for left-turn lanes approaching arterial streets, and 150-ft storage lanes for those approach-

Table 8. Recommended storage lengths for arterials from *Access Management Manual* equation and *NCHRP Report 457* equations with revised critical gap.

Left-Turn Volume (veh/hr)	Storage Length, Rounded Up to Nearest 25-ft Increment (ft)						
	Storage Lengths from Other Manuals for Comparison		Storage Lengths Calculated from Equations [b] Documented in *NCHRP Report 457* Using Revised Critical Gaps and 0.005 Probability of Overflow				
	Green Book Procedure (k=1)[a]	Equation (k=2)[a]	Opposing Volume (veh/hr)				
			200	400	600	800	1000
Critical Gap = 5.0 sec, Follow-Up Gap = 2.2 sec (Represents the 50th Percentile Critical Gap Found in Field Studies)							
40	75	75	50	50	50	50	50
60	50	100	50	50	50	50	50
80	75	150	50	50	50	50	50
100	100	175	50	50	50	50	75
120	100	200	50	50	50	75	75
140	125	250	50	50	50	75	75
160	150	275	50	50	75	75	100
180	150	300	50	50	75	75	100
200	175	350	50	75	75	100	125
220	200	375	50	75	75	100	125
240	200	400	75	75	100	125	150
260	225	450	75	75	100	125	175
280	250	475	75	75	100	125	175
300	250	500	75	100	125	150	200
Critical Gap = 6.25 sec, Follow-Up Gap = 2.2 sec (Represents the 85th Percentile Critical Gap Found in Field Studies, 85th Percentile 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
0	250	500	75	100	150	225	525

[a, b] See Table 7 for equations.
This table assumes 25 ft per vehicle spacing. Table 4 provides other suggested spacing lengths based on percent trucks.

ing collector streets and most local streets, with a minimum length of 100 ft at local streets and minor driveways.

The designer should also consider that if the appropriate design vehicle is a truck or other large vehicle instead of a passenger car, the minimum storage length must be extended accordingly.

Sight Distance

All locations along a roadway from which vehicles are permitted to turn left across opposing traffic, including intersections and driveways, should have sufficient sight distance to accommodate the left-turn maneuver. Left-turning drivers need sufficient sight distance to decide when it is safe to turn left across the lane(s) used by opposing traffic. Sight distance design should be based on a left turn by a stopped vehicle since a vehicle that turns left without stopping needs less sight distance. Based on that scenario, AASHTO recommends sight distance values for left turns from a major road as shown in Table 9. These distances are for a passenger car crossing one lane of opposing traffic; the *Green Book* provides adjustment factors for other vehicles and lane configurations.

If stopping sight distance has been provided continuously along the major road and if sight distance has been provided for each minor-road approach, sight distance will generally be adequate for left turns from the major road. Therefore, no separate check for sight distance for left turns may be needed. However, at three-leg intersections or driveways located on or near a horizontal curve or crest vertical curve on the major road, the availability of adequate sight distance for left turns from the major road should be checked. In addition, the availability of sight distance for left turns from divided highways should be checked because of the possibility of sight obstructions in the median.

Table 9. Intersection sight distance—left turn from major road (*5*).

Design Speed (mph)	Stopping Sight Distance (ft)	Intersection Sight Distance	
		Calculated (ft)	Design (ft)
15	80	121.3	125
20	115	161.7	165
25	155	202.1	205
30	200	242.6	245
35	250	283.0	285
40	305	323.4	325
45	360	363.8	365
50	425	404.3	405
55	495	444.7	445
60	570	485.1	490
65	645	525.5	530
70	730	566.0	570
75	820	606.4	610
80	910	646.8	650

If an intersection is known to have a substantial portion of older drivers, the FHWA *Highway Design Handbook for Older Drivers and Pedestrians* (*15*) makes two recommendations for accommodating the slower decision times of older drivers:

1. Where sight-distance requirements incorporate a perception-reaction time (PRT) component, it is recommended that a PRT value of no less than 2.5 sec be used to accommodate the slower decision times of older drivers.
2. Where a gap model is used to determine sight-distance requirements, it is recommended that a gap of no less than 8.0 sec, plus 0.5 sec for each additional lane crossed by the turning driver, be used to accommodate the slower decision times of older drivers.

Median Design

Special concern should be given to median width. *NCHRP Report 375* (*16*) has found that most types of undesirable driving behavior in the median areas of divided highway intersections are associated with competition for space by vehicles traveling through the median in the same direction. The potential for such problems is limited where crossroad and U-turn volumes are low, but may increase at higher volumes. In the study, at rural unsignalized intersections, the frequency of undesirable driving behavior and crashes decreased as the median width increased; this implies that medians should be as wide as practical. Also, the frequency of undesirable driving behavior increased as the median opening length increased.

At intersections where it is necessary to store turning vehicles in the median, a width of 12 to 30 ft provides protection for left-turning vehicles. In many cases, the median width at rural unsignalized intersections is a function of the design vehicle selected for turning and crossing maneuvers. Where a median width of 25 ft or more is provided, a passenger car making a turning or crossing maneuver has space to stop in the median area. Medians less than 25 ft wide should be avoided at rural intersections because drivers may be tempted to stop in the median with part of their vehicles unprotected from through traffic. The school bus is often the largest vehicle to use the median roadway frequently. The selection of a school bus as the design vehicle results in a median width of 50 ft. Larger design vehicles, including trucks, may be used at intersections where enough turning or crossing trucks are present; median widths of 80 ft or more may be needed to accommodate large tractor-trailer trucks without encroaching on the through lanes of a major road.

The form of treatment given the end of the narrowed median adjacent to lanes of opposing traffic depends largely on the available width. The narrowed median may be curbed to delineate the lane edge; to separate opposing movements; to provide space for signs, markers, and luminaire supports; and to protect pedestrians. To serve these purposes satisfactorily, a minimum narrowed median width of no less than 4 ft is recommended and one of 6 to 8 ft wide is preferred. These dimensions can be provided within a median 16 to 18 ft wide using a turning-lane width of 12 ft. Additional information on median design is in the AASHTO *Green Book* (*5*) and the Institute of Transportation Engineers (ITE) *Urban Street Geometric Design Handbook* (*17*).

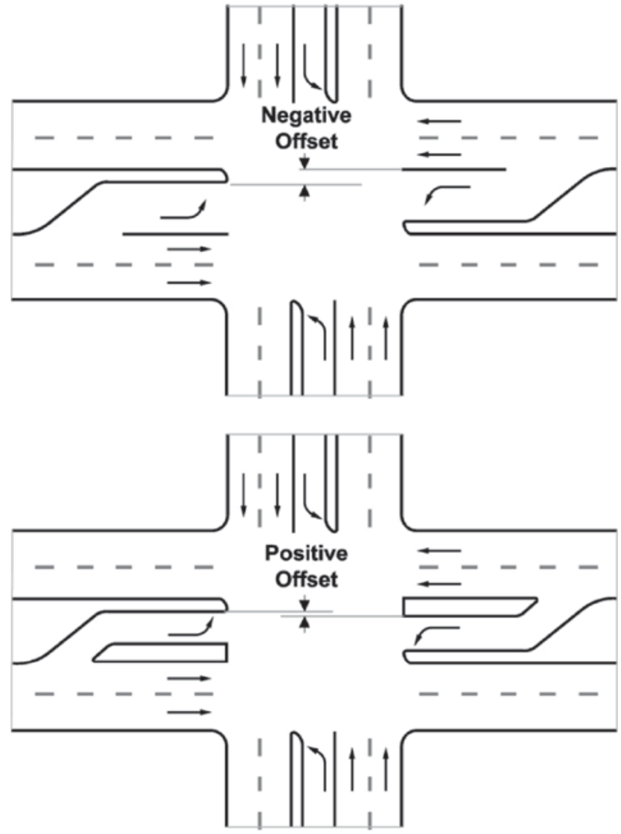
Channelization and Offset

Vehicles in opposing left-turn lanes can limit each other's views of opposing traffic, and wide medians can affect turning drivers' ability to accurately judge available gaps in opposing traffic. If large trucks are in one or both directions, there is a substantially higher need to provide offset to allow motorists to better see past the stopped truck in the opposite direction. The restriction on the sight distance is dependent on the amount and direction of the offset between the opposing left-turn lanes. For medians wider than about 18 ft, it is desirable to offset the left-turn lane so that it will reduce the width of the divider to 6 to 8 ft immediately before the intersection, rather than to align it exactly parallel with and adjacent to the through lane. This alignment 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 providing improved visibility of opposing through traffic. The advantages of offsetting the left-turn lanes are:

- Better visibility of opposing through traffic;
- Decreased possibility of conflict between opposing left-turn movements within the intersection; and
- More left-turn vehicles served in a given period of time, particularly at a signalized intersection (*16*).

Offset is measured between the left edge of a left-turn lane and the right edge of the opposing left-turn lane as shown in Figure 10 (*7, 17*). This left-turn lane configuration is referred to as a parallel offset left-turn lane and is further illustrated in Figure 11A. Parallel offset left-turn lanes may be used at both signalized and unsignalized intersections.

An offset between opposing left-turn vehicles can also be achieved with a left-turn lane that diverges from the through lanes and crosses the median at a slight angle. Figure 11B illustrates a tapered offset left-turn lane of this type. While used primarily at signalized intersections, tapered offset left-turn lanes provide the same advantages as parallel offset left-turn lanes in reducing sight distance obstructions and potential conflicts between opposing left-turn vehicles and in increasing the efficiency of signal operations. Tapered offset left-turn lanes are normally constructed with a 4-ft nose between the left-turn lane and the opposing through lanes.



(Source: *Urban Intersection Design Guide*, FHWA/TX-05/04365-P2. Reproduced with permission from author.)

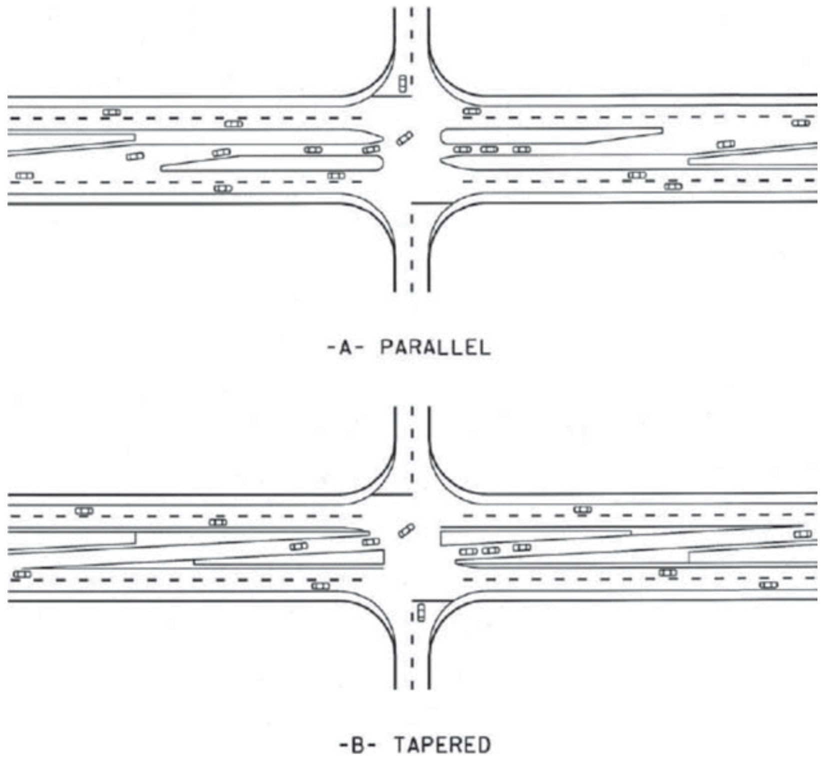
Figure 10. Negative and positive offsets of left-turn lanes (7).

This type of offset is especially effective for turning radius allowances where trucks with long rear overhangs, such as logging trucks, are turning from the mainline roadway.

Parallel and tapered offset left-turn lanes should be separated from the adjacent through traffic lanes by painted or raised channelization. A long separation between the left-turn lane and the adjacent through traffic lane results in a "single point" entry (transition) into the left-turn lane. This can result in excessive deceleration in the through traffic lane. An alternative is to provide a parallel left-turn deceleration lane adjacent to, and separated from, the through lane marked by striping (see Figure 11B for an illustration) or to use an island (separation) between the left-turn lane and the adjacent through lane in proximity to the intersection to "shift" the turning vehicle to the left and create a positive offset.

Bypass Lanes

Figure 9 shows a particular type of left-turn lane installation, with direct entry into the turning lane. This alignment allows the through driver to change lanes to avoid the left-turning vehicle



(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 11. Parallel and tapered offset left-turn lanes (5).

and continue through the intersection. It is commonly called a bypass lane. This alignment may be used where right-of-way is constrained but a left-turn lane is warranted. The bypass lane alignment has no shadowing for the left-turn lane; thus, the bay taper shown in Figure 7 and Figure 8 is not necessary, which shortens the length of additional right-of-way needed.

Though the bay taper length is not needed on a bypass lane, it is still necessary to provide a transition taper to guide through traffic around the left-turn lane. The length of the transition taper for a bypass lane is typically shorter than that used for a shadowed or partially shadowed lane when through traffic is shifted to the right, such as those shown in Table 5. Some states refer to this length as the deceleration transition, because the shorter distance still requires drivers to decelerate from the prevailing operating speed. For example, Indiana (*18*) uses 300 ft for a design speed of 50 mph or greater and Minnesota (*19*) uses 1:15 (180 ft for a 12-ft bypass lane).

Depending on the configuration of the intersection, it is also necessary for a bypass lane to have another transition taper as the through traffic returns to its original alignment. This transition may also be referred to as the acceleration transition, and it is commonly the same length as the transition taper on the approach to the intersection (see Figure 12).

Agencies may consider the use of bypass lanes at "T" intersections in undeveloped areas when left-turn lane warrants are met but the installation of a left-turn lane is not practical. Some states do not allow informal passing on the right or driving on the shoulder; constructing the additional width for through vehicles provides a legal means of passing slowed or stopped left-turning vehicles.

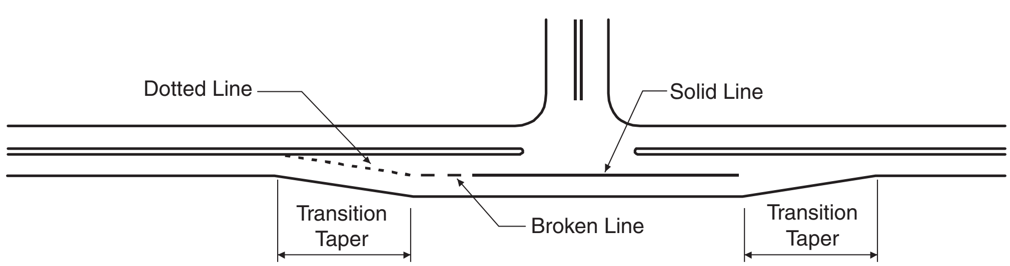


Figure 12. Example of bypass lane with markings.

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 dotted-line pavement markings that have a much shorter stroke length and shorter spacing than broken lane markings that permit passing. This pavement marking is referred to as a dotted line and is illustrated in Figure 12. The pavement marking reinforces the message that through drivers must change lanes but still permits left-turn drivers to travel straight into the turning lane.

Pedestrian Storage

For urban and suburban locations, as well as rural locations with high pedestrian crossing volumes, adequate storage must be provided for pedestrians in the median adjacent to the left-turn lane. When pedestrian storage is provided, it must also be fully accessible by pedestrians with disabilities. According to the *Revised Draft Guidelines for Accessible Public Rights-of-Way* (*8*), medians and pedestrian refuge islands in crosswalks must comply with the dimensions of a pedestrian access route and connect to each crosswalk. A pedestrian access route must have a continuous and unobstructed clear width of at least 4 ft, exclusive of the width of the curb. The cross slope of a pedestrian access route must be no more than 2 percent.

Medians and pedestrian refuge islands must be at least 6.0 ft in length in the direction of pedestrian travel, and they must have detectable warnings at curb ramps and blended transitions. Detectable warnings at cut-through islands must be located at the curb line in line with the face of the curb and must be separated by a 2.0-ft minimum length of walkway without detectable warnings. Where the island has no curb, the detectable warning must be located at the edge of the roadway.

CHAPTER 4

Traffic Controls and Illumination

Introduction

Traffic controls are important complements to the geometric design of unsignalized intersections. Signs, signals, and markings should conform to guidance set forth in the *Manual on Uniform Traffic Control Devices* (MUTCD) (*20*). State and local standards for roadway and intersection illumination should reflect AASHTO and Illuminating Engineering Society of North America (IESNA) guidelines (*21*). Consistency in the type, size, shape, messages, and placement of devices is essential. In this chapter, there are three categories of devices considered:

- Signs, which include regulatory, warning, and information signs;
- Pavement markings, which include centerlines, lane lines, stop bars, and crosswalk striping; and
- Illumination, which includes intersection and approach lighting.

Signs

Traffic signs are used to convey essential information along streets and highways and at intersections. Sign types, shapes, sizes, symbols, colors, messages, illumination, reflectorization, and placement should be consistent with guidelines set forth in the MUTCD (*20*) and state and local requirements. Additional guidance is given in the ITE *Traffic Control Devices Handbook* (*22*) and the ITE *Traffic Engineering Handbook* (*23*).

Signs should be used only where warranted based on engineering judgment or studies. Signs should be coordinated with the geometric design and with the associated pavement markings. Excessive signing should be avoided. However, where the population of older drivers is significant, some redundant signing may be applicable.

Sign Types

Traffic signs are classified as regulatory, warning, or guide signs:

- **Regulatory signs** give notice of traffic laws and regulations. Common signs used at unsignalized intersections include STOP and YIELD signs. Where special turn lanes are provided, lane-use signs are normally installed. ONE WAY and DO NOT ENTER signs are used where some streets operate one way.
- **Warning signs** give notice of situations that might not be readily apparent. The Intersection Ahead sign may be used on approaches to intersections.
- **Guide signs** show route designations, destinations, distances, and services. Street Name signs are guide signs that are commonly installed at intersections.

Sign Design

Sign size, design, and location are governed by Part 2 of the MUTCD (*20*). Signs must be sufficiently large, legible, readable, and properly placed for both day and night visibility. The general principles of sign size, shape, color, and messages are discussed in Chapter 2A of the MUTCD (*20*). Larger signs may be used where increased visibility is essential. FHWA's *Standard Highway Signs* manual (*24*) provides detailed sign layouts for standard signs contained in the MUTCD.

Key features of commonly used intersection and intersection-related signs are listed in Table 10. The table gives the sign type, MUTCD number, size, shape, and color. Advance traffic control signs, such as STOP Ahead or YIELD Ahead, may be used on approaches to intersections that are controlled by these devices. The MUTCD requires the use of these signs where the primary intersection traffic control devices are not visible for a sufficient distance.

Table 10. Characteristics of signs commonly used at unsignalized intersections and approaches (adapted from *20*).

Type	Sign	MUTCD Number	Shape	Typical Size (Inches)	Background Color	Text Color
Regulatory	STOP	R1-1	Octagon	30×30 or 36×36	Red	White
	YIELD	R1-2	Equilateral triangle	36×36×36	Red	White
	No Turns	R3-1 through R3-4	Rectangular	24×24	White	Black
	Lane Controls	R3-5 through R3-8	Rectangular	30×36	White	Black
	DO NOT ENTER	R5-1	Circle on square plate	30×30	Red	White
	ONE WAY	R6-1 R6-2	Rectangular	36×12 24×30	White	Black
Warning	Stop Ahead	W3-1a	Rectangular	24×30	Yellow	Black
	Yield Ahead	W3-2	Rectangular	24×30	Yellow	Black
	Crossroad	W2-1	Rectangular	24×30	Yellow	Black
	Intersection Ahead	W2-1	Rectangular	24×30	Yellow	Black
Guide	Street Name	D-3	Rectangular	Varies	Green	White
	Advance Street Name	W16-8P, W16-8aP	Rectangular	Varies	Green	White

Word messages should be as brief as possible. Abbreviations should be kept to a minimum. Punctuation (e.g., periods, apostrophes, ampersands) should not be used; word messages should contain only characters that are letters, numerals, or hyphens unless other characters are necessary to avoid confusion. Letter sizes should be large enough to be readable at appropriate distances in advance of sign locations. The MUTCD uses a guideline of 30 ft of distance for each 1 inch of letter height. The *Highway Design Handbook for Older Drivers and Pedestrians* (*15*) recommends a ratio of 33 ft to 1 inch of letter height for enhanced visibility. Increases above standard sizes should be used where greater legibility or emphasis is needed. Wherever practical, the overall sign dimensions should be increased in 6-inch increments (*15*).

Most standard highway signs use all capital letters for the sign legend. However, a combination of uppercase and lowercase letters provides greater legibility for destination names on guide signs. Allowing drivers to read the unique footprint of the words displayed in uppercase and lowercase letters increases accuracy, viewing distance, and reaction time compared to words in all capital letters. When a mixed-case legend is used, the height of the lowercase letters shall be three-fourths of the height of the initial uppercase letter.

Placement

Sign location and placement should reflect several basic objectives:

- Location and placement should be consistent along highways and at intersections.
- Signs should be located within the approaching driver's cone of vision.
- Signs should not be located within the traveled way (an exception is some pedestrian crossing signs) or pedestrian areas.
- Signs should not interfere with drivers' visibility of hazards, especially intersections or driveways.

The 2009 MUTCD contains guidelines for advance placement of warning signs in Table 2C-4 (*20*).

Longitudinal Placement

Signs should be placed at locations where there are no sight obstructions between the sign location and the intended point of observation. Locations that should be avoided include dips and hillcrests, trees and foliage, other signs, and parked vehicles. Sign placement should not create an obstruction to pedestrians, bicyclists, or drivers' visibility of hazards (near intersections or driveways), and should not be vulnerable to roadside splatter or snow-plowing operations.

Using an excessive number of signs should be avoided. Regulatory and warning signs should be used conservatively because these signs, if used to excess, tend to lose their effectiveness. If used, route signs and directional guide signs should be used frequently because their use promotes efficient operations by keeping road users informed of their location.

Signs should not interfere with each other and should be placed where they are not obstructed by poles or trees. The longitudinal placement depends on the type of sign, nature of the message, and desired motorist response, but some general guidelines are as follows:

- Regulatory signs should be placed at the locations where the regulation begins.
- Warning signs should be placed sufficiently in advance of the hazard, where drivers can take appropriate corrective actions.
- Information signs should be placed near the location where they apply. Specific locations vary with the type of message offered.
- Successive signs on approaches to intersections should be spaced as widely as possible. Signs requiring separate decisions by the road user should be spaced sufficiently far apart for the appropriate decisions to be made. One of the factors considered when determining the appropriate spacing is the posted or 85th percentile speed.

Signs should be individually installed on separate posts or mountings except where:

- One sign supplements another;
- Route or directional signs are grouped to clarify information to motorists;
- Regulatory signs that do not conflict with each other are grouped, such as turn prohibition signs posted with ONE WAY signs or a parking regulation sign posted with a speed limit sign; or
- Street Name signs are posted with a STOP or YIELD sign.

Lateral Clearance

Signs are generally located as far as practical from the traveled way while still providing good visibility. They should not protrude into pedestrian areas. Some guidelines include:

- Signs on rural roadways should be placed at least 6 ft beyond the shoulder or at least 12 ft from the edge of the traveled way, whichever is greater.
- On streets and roadways with curbs, signs should be at least 2 ft beyond the face of the curb. A clearance of 1 ft is permissible where sidewalk width is limited.

Vertical Clearance and Mounting Height. The vertical clearance (of the bottom of the sign) must be at least 7 ft above the surface in urban areas to provide visibility above parked cars and not pose a hazard to pedestrians. The minimum height is 5 ft in rural areas. The vertical clearance on overhead-mounted signs is at least 17 ft above the edge of pavement. Both span-wire and mast-arm cantilevered mountings are permissible. Street Name signs may be placed above a regulatory or STOP sign with no vertical separation.

Application Guidelines

The application of specific signs should be consistent with requirements set forth in the MUTCD and state/local requirements. Relevant application guidelines for regulatory, warning, and information signs follow:

- **Regulatory signs** should be installed at or near where the regulations apply. They inform users of traffic laws, regulations, or restrictions that are applicable for the given intersection and approaches.
- **Warning signs for intersection approaches** are commonly used upstream of an intersection to advise motorists of upcoming intersections. They include signs advising motorists of the Intersection Ahead and of a STOP Ahead or YIELD Ahead. These signs are placed a sufficient distance ahead to enable motorists to take appropriate actions. Typical placement of advance traffic control signs is shown in the MUTCD. Various signs are used to give direction to pedestrians and give warning to motorists. A common application is to provide Pedestrian Crossing signs along roadways where pedestrian movements might occur.
- **Guide signs** give road users information in the most simple and direct manner possible. They:
  - Direct road users along streets and highways and inform them of intersecting routes;
  - Direct road users to cities, towns, villages, and other important destinations; and
  - Identify nearby rivers and streams, parks, forests, and historical sites.

  Conventional guide signing gives highway class, number, and cardinal direction. This information is provided at key intersections. Pedestrian information, usually in advance of intersections, is sometimes repeated at intersections. Street Name signs should be installed at all intersections in urban and suburban areas. They should be installed in rural areas to identify important roads not otherwise signed. In residential areas, at least one Street Name sign should be mounted at each intersection; their faces should be mounted parallel to the streets they name. Signs may also be mounted overhead on the far side of an intersection. Advance Street Name signs using white letters on a green background may be installed near exclusive turn lanes, as well as on approaches to major highways. In business districts and on principal arterials, Street Name signs should at least be placed on diagonally opposite corners so they will be located on the far right side of the intersection along the major street.

Example applications are available in the MUTCD (*20*). The Texas Department of Transportation *Sign Crew Field Book* (*25*) provides specific guidance for intersection signing developed to provide field sign personnel with information beyond that contained in the Texas MUTCD (*26*) to improve statewide uniformity in the placement of traffic signs.

Pavement Markings

Overview

Pavement markings are a key element in roadway design and operations. They are an effective and efficient way to improve clarity and safety. They are located directly in the driver's cone of vision, they provide continuous information, and they are an important complement to signs. They have a dual purpose of guiding users and optimizing roadway efficiency. They are normally used to supplement signs and signals and to convey messages that might not otherwise be understood.

Typical markings are attached to the pavement surface and convey information by color, line design, words, letters, symbols, and patterns that regulate, warn, or guide users. Some examples of the uses of pavement markings include:

- No passing zones (indicate regulations),
- Stop bars (supplement or reinforce the message of other devices),
- Lane lines (guide road users), and
- Text markings such as SIGNAL AHEAD or RR XING (warn users).

In addition to markings, post-mounted delineators, object markers, and colored pavements can also be considered part of the marking system.

Markings define edge lines, travel lanes, stop lines, and crosswalks. They are sometimes used in channelizing intersections and as painted islands that separate opposing traffic streams.

Markings, like other traffic controls, should comply with the MUTCD and state manuals. Uniformity and clarity are essential to convey a consistent message to drivers that meets their expectations and effectively communicates the necessary information.

Types of Markings

For the purposes of these guidelines, pavement markings are classified as longitudinal (along the roadway), transverse (perpendicular to the roadway), and on-pavement messages.

- Longitudinal markings include edge lines, lane lines, centerlines (or painted median islands), lane transition lines, no passing zones, and curb delineations.
- Transverse markings include stop lines and crosswalks.
- Messages on pavement include markings such as arrows, bus or light-rail transit lane symbols, high-occupancy vehicle (HOV) symbols, or text.
- White is used for most pavement markings. In the United States, centerlines that separate opposing directions of travel are yellow.
- Other colors (e.g., red, blue, or purple) are sometimes used to indicate restrictions or special conditions, but they are not common at unsignalized intersections.
- Black may be used in combination with white or light-colored markings to improve the contrast with light-colored pavements.

The basic types of markings, sample applications, and their colors are set forth in Table 11.

Applications

The applications of pavement markings at unsignalized intersections are generally straightforward. They include the delineation of through and turning lanes, provision of crosswalks, and possible provision of broken lines (commonly known as "skip stripes" or "cat tracks") to guide left turning. The MUTCD (*20*) shows examples of typical markings at intersections.

Table 11. Overview of pavement markings (*7*).

Type	Application	Color	Markings
Longitudinal	Edge lines	White	Solid
	Lane lines	White	Usually dotted
	Center lines	Yellow	Solid or dotted
	Lane transition lines	White	Dotted
	No passing zones	Yellow	Solid
Transverse	Stop lines	White	Solid
	Pedestrian crosswalks	White	Various patterns
Messages on Pavement	Directional arrows (optionally accompanied by text)	White	Symbol (and optional text)
	HOV or bus symbols	White	Symbol
	PED XING	White	Text

Pavement markings should be integrated into the physical design. Key elements include:

- Through and left-turn lane widths,
- Required left-turn lane storage lengths, and
- Transition markings for lanes guiding through traffic around left turns and shadowing the left-turn lane.

Specific pavement markings depend upon how the left-turn lanes are provided. Following are three example cases:

- Where the left-turn lanes can be provided by utilizing a central point (or raised island) median, there is no alignment change for the through travel lanes, as shown in Figure 6.
- In most cases, it is necessary to widen roadways or eliminate on-street parking to provide space for left-turn lanes. The example in Figure 7 diverts the through travel lanes around a fully "shadowed" left-turn lane. This concept has application where there are high approach speeds.
- The example in Figure 9 offsets the through travel lane and allows direct entry into the left-turn lane. This concept is commonly used in many cities (e.g., Chicago, New Haven, Toronto).

Illumination

Overview

Roadway and intersection lighting is an important complement to traffic controls in improving both vehicle and pedestrian operations and safety. They provide for the visual needs of motorists, bicyclists, and pedestrians. Dark conditions make it difficult for drivers to make correct decisions with adequate time. Where adequate lighting is provided, sudden braking and swerving are reduced, and visibility of pavement markings and signing is enhanced.

This section gives a broad overview of intersection lighting. It contains discussion of the types of systems, how lighting can be designed, and where roadway and intersection lighting should be placed. Further details are set forth in the ITE *Traffic Engineering Handbook* (*23*), the AASHTO *Roadway Lighting Design Guide* (*27*), and the *IESNA Lighting Handbook* (*21*).

Types of Illumination

Street and roadway lighting units comprise the lamp, ballast, and luminaire. The following factors influence their selection (*21*):

- Lamp lumen output (lamp size),
- Lamp life,
- Lamp lumen depreciation,
- Ambient temperature range in the area,
- Cost (lamp and luminaire),
- Lamp restrike time,
- Luminaire light distribution,
- Physical size (lamp and luminaire),
- Physical durability (lamp and luminaire),
- Lamp color, and
- Energy consumption.

Low-pressure sodium (LPS), high-intensity discharge (HID) (commonly known as mercury), metal halide, and high-pressure sodium (HPS) light sources are frequently used in today's roadway lighting systems. Other types include incandescent and fluorescent. Light-emitting diodes (LEDs) are mainly used in traffic signals and are under field tests for street lighting. Homburger et al. (*28*) described a number of characteristics of these lamps (listed in Table 12).

- **The incandescent or filament lamp** was the most commonly used for many years. It was inexpensive, simple, and easy to install. It produced pleasing color rendition, and its small size permitted good light control with a reasonably sized fixture. However, its low efficacy and short-rated life have made it undesirable for new installations.

Table 12. Roadway lighting lamp characteristics (*28*).

Type of Lamp	Initial Light Output (1000 Lumens)	Approximate Efficacy (Lumen/Watt)[a]	Approximate Lamp Life (1,000 Hours)[b]
Incandescent	0.6–15	9.7–17.4	2–6
Fluorescent	6.8–14	61–72	10
Clear Mercury	3.7–57	37–57	18–28
Phosphor-Coated Mercury	4.0–63	40–63	18–28
Metal Halide	34.0–100	85–100	10–15
High-Pressure Sodium	9.5–140	95–140	15–28
Low-Pressure Sodium	1.8–33	100–183	10–18

[a] Except for incandescent, these values exclude wattage losses due to ballast.
[b] Number of hours for a group of lamps where 50 percent will remain in operation; based on 10 hours of operation per start, except 3 hours per start for fluorescent lamps.

- **The fluorescent lamp** is no longer used for new roadway lighting installations but is still utilized for tunnel and sign lighting. Its large size makes it difficult to obtain good light control in a reasonably sized luminaire. The lamp requires ballast, and its light output is affected by low temperature more than other lamps. An advantage is the broad light patterns that it provides on wet streets.
- **The mercury lamp**, developed in the 1930s, replaced the incandescent lamp in popularity. Its initial cost is higher, and it requires ballast, but its higher efficacy and long life make it more attractive than the incandescent lamp. The blue-white color of the clear lamp is generally acceptable, and the arc tube size provides a light source that is small enough to permit good light control.
- **The metal halide lamp**, a type of mercury lamp, has arc tubes that also contain certain metal halides without the use of a phosphor-coated bulk. The lamp produces a white light bulb that is pleasing to motorists and pedestrians. The light source is that of the arc tube, permitting good light control in the same feature used for clear mercury lamps. They work well in high-mast lighting. The lamps have a wide selection of sizes, good lamp life, and a compact size; they are easily optically controlled.
- **The HPS lamp** is characterized by a golden-white color output. HPS lamps are normally operated with special ballasts that provide the necessary voltage to start the lamp. Some of the newer HPS lamps include improved color rendition, internal starting devices that operate with mercury or metal halide lamp ballasts, dual arc tube or "standby" lamps that provide light as soon as power is restored after a momentary power interruption, and a rated life of 40,000 hours.
- **The LPS lamp** is characterized by a monochromatic bright yellow color output. It requires special ballasts and increases materially in size as the wattage increases; the 185-W lamp is 1120 mm long. This large size makes it difficult to obtain good light control in a reasonably sized fixture. The poor color rendition of the LPS lamp previously made it unpopular for use in other than industrial or security applications. However, the potential benefits of energy conservation produced by the high efficacy of the lamp have resulted in its increasing acceptance for lighting both commercial and residential areas.
- **LEDs** have the advantages of good light distribution and long service life (100,000 hours). They are used for traffic signal displays and offer promise, but are not yet widely used for street and roadway lighting. Good heat management is essential.
- **Ballast** is required for HID and fluorescent lamps. The ballast provides the proper starting and operating voltage and wave form, and limits the operating current to the proper value; for certain types of lamps it supplies the necessary cathode heater voltage. Ballast may be located in the luminaire housing, in the pole base, or in an underground pull box, the most popular location being in the luminaire housing.

Intensity

Roadway and intersection lighting design and placement should consider the visual capabilities of motorists and pedestrians, roadway geometry, traffic volumes, operating speeds, energy consumption, and environmental features.

The principal steps in the lighting design process are summarized as follows [further details are contained in the ITE *Traffic Engineering Handbook* (*23*)]:

1. Survey and evaluate existing roadway, traffic, and environmental conditions.
2. Classify roadway types, pavement reflections, and the surrounding environmental features.
3. Select the required minimum illumination levels for the roadway or intersection under consideration.
4. Acquire photometric and related information for the lamps and luminaires to be used. This information includes watts, lumens, light distribution, and lumen depreciation factors.
5. Estimate light loss factors for luminaire dirt depreciation, effects of ambient temperature, possible field adjustments to laboratory tests, operating voltage variations, and lamp/ballast combined luminaire depreciation factors.
6. Establish desired luminaire mounting heights. This normally depends on the lumens provided:
   - ≤ 20,000 lumens ≤ 35 ft
   - 30,000–45,000 35–45 ft
   - 45,000–90,000 45–60 ft
7. Compute the desired luminaire spacing for the required illumination levels.
8. Compute the uniformity ratio (the ratio of the average maintained intensity to the minimum). Repeat steps 4 through 7 as needed.
9. Also, transitional lighting at the beginning and end points of continuous lighting systems should be considered. This may not be necessary when illumination is being provided for an isolated location.

Location

Most urban and suburban street intersections are illuminated. In rural areas, however, intersection illumination is often lacking. Accordingly, many states have developed guidelines for when intersection lighting is warranted. Improved nighttime safety is usually a main consideration, but the practitioner should consult the guidance documents that are in effect for that jurisdiction to learn about requirements and guidelines that are specific to the state, county, or city in which the intersection is located.

The following factors should be considered in assessing intersections for lighting:

- Intersection location (urban, suburban, or rural);
- Functional classification of the approach roads;
- Intersection visibility, especially nighttime approach visibility;
- Street geometry, including merging lanes, curves, and grades;
- Intersection channelization, including the presence of physical medians;
- Presence of lighting on approaches;
- Traffic volumes and speeds;
- Pedestrian and bicycle volumes; and
- Intersection safety (night-to-day crash ratios).

Based on current practices in the United States, and a review of existing guidance documents and previous research, several general guidelines are established regarding where lighting should be considered for rural intersections. An intersection approach with one or more of the following characteristics is a good candidate for lighting:

- The intersection geometry is complex and includes raised channelization.
- The intersection sight distance is less than ideal (e.g., a roadway located on a curve).
- Nighttime crash rates are high over a multiyear period, and the nighttime-to-daytime crash rate ratios are high.
- One or more approaches to the intersection are already illuminated.

Provision for Future Signalization

Though the material in this *Design Guide* is focused on unsignalized intersections, it is prudent to make provisions to add traffic signals in the future, particularly in areas of expected growth and development on suburban fringes and in areas described by a city's comprehensive development plan. This section discusses a selection of key items to consider when planning for future signalization.

Power

The need to provide electrical power to the intersection must be a primary consideration. Typically there are procedures in place for extending the power grid to new locations, and it is just a matter of paying for the cost of new power lines and, depending on the location, conduits or poles to carry the lines to the intersection. If lighting is already included in the left-turn lane installation, then those procedures will be followed, and power will already be available at the intersection when signals are added.

Location of Signal Poles and Controller Cabinet

This provision dovetails with that of providing power. It is common to install roadway lighting on signal poles. Conversely, if the lighting is installed first at a location where signals are planned, the luminaire poles can be placed such that they can be used for pole-mounted or mast-arm signal heads when the traffic signals are installed. When lighting is not installed, it is important to maintain the necessary clear space within the right-of-way (or acquire additional right-of-way) to accommodate the future installation of the signal poles and controller cabinet.

Vehicle Detection

It is common to use video cameras to detect vehicles awaiting a green indication at a signalized intersection, and these cameras can be placed on the signal pole or mast arm. However, if it is anticipated that other types of detection will be used, particularly some form of in-pavement sensor, it may be beneficial to install much of the hardware for that detection system while the intersection is under construction. This will prevent the need to go back into the new pavement at a later date when volumes are likely higher and the impact on operations at the intersection will be higher. Similarly, an important consideration in addition to the sensors themselves is the installation of conduit and junction boxes. If conduit of the proper size to accommodate a future signal is not installed with the project, a future project will be required to create a trench for the conduit at a later date, which will likely eradicate any in-pavement sensors previously installed.

Access Management

At unsignalized intersections of lower volume, adjacent developments may install (or request installation of) access points near the intersection. These points of access may function adequately under initial traffic conditions, but as volumes rise, operations and level of service will likely suffer, and safety problems may develop. If future signalization is anticipated, it is prudent to prohibit access points in close proximity to the intersection. This will eliminate the later need to close or move a driveway or to accommodate access during future growth or construction.

CHAPTER 5

Design Examples

Introduction

This chapter includes 11 design examples and case studies, based on actual locations at which left-turn treatments were considered, evaluated, and/or installed. These case studies include:

1. Installation of exclusive left-turn lanes at an unsignalized intersection in a suburban fringe area.
2. Installation of an exclusive left-turn lane at an unsignalized intersection between a local street and a state highway.
3. Addition of a left-turn lane as part of a "road diet" treatment (conversion of a four-lane cross section to a three-lane cross section).
4. Installation of left-turn lanes at an unsignalized rural intersection between a state highway and a local road.
5. Installation of unsignalized "J-turn" intersections along a state highway.
6. Installation of a left-turn passing blister (i.e., bypass lane) at an unsignalized intersection between a local street and a state highway.
7. Installation of left-turn lanes at an unsignalized rural intersection between a state highway and a county road.
8. Follow-up traffic studies to verify the need for an unsignalized left-turn lane in conjunction with a proposed development.
9. Installation of left-turn lanes at unsignalized intersections in conjunction with a proposed development.
10. Installation of an exclusive left-turn lane at an unsignalized suburban intersection between a state highway and a local street.
11. Installation of exclusive left-turn lanes and a traffic signal at an unsignalized intersection between a local street and a state highway.

These design examples are intended to provide the practitioner with real-world scenarios that are similar to those the practitioner would actually encounter, along with possible solutions and the methods by which those solutions can be evaluated and installed.

Design Application #1: Installation of Exclusive Left-Turn Lanes at an Unsignalized Intersection in a Suburban Fringe Area

(*Note: The intersection location in this design application has been made anonymous at the request of the contributing agency. Street names have been changed.*)

Context

In a rapidly developing area, citizens are concerned with the number of cars using the shoulder to pass slow-moving left-turning vehicles on State Highway 41 (see Figure 13). The area is currently considered rural; however, the anticipated development within the next 5 years will change the performance of the roadway. Ultimately, State Highway 41 will provide more access and less mobility. It has already started evolving into a suburban high-speed arterial. Designers noted that the state's *Access Management Manual* contains criteria on connection spacing for state highways. They decided to use the warrants developed as part of NCHRP Project 3-91 and the state's *Access Management Manual* as part of their traffic operations evaluation, to determine whether a left-turn lane was appropriate at the intersection with David Drive.

The intersection of State Highway 41 and David Drive is a T-intersection. The information known for this site includes:

- State Highway 41 is a two-lane roadway.
- Peak-hour turning-movement counts are shown in Figure 14.
- The 85th percentile speed is 59 mph.
- Posted speed is 55 mph.

Figure 13. Example of vehicle using shoulder to pass left-turning vehicle.

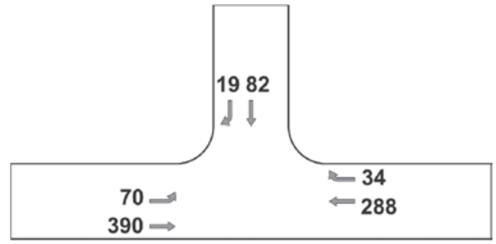


Figure 14. Peak-hour turning-movement count.

- Grades at the intersection are level.
- Lane widths are 12 ft.
- Shoulder widths on the major road are 10 ft.
- No other left-turn bay is within 1 mile of the site.

Design Considerations and Analysis

Consideration of Turning and Through Volumes

The primary consideration for the department of transportation (DOT) is the increasing volumes on the major highway, both turning and through movements. In this case, the peak-hour advancing volume from Figure 14 is (390 + 70) = 460 veh/hr, and the opposing volume is (288 + 34) = 322 veh/hr. The resulting overall volume on the major highway is (460 + 322) = 782 veh/hr, or 391 vehicles per hour per lane.

Using Table 1 in Chapter 2, the designer can determine the recommended volumes for a left-turn lane on a rural highway. The table is reproduced as Table 13, and the relevant volumes are highlighted. The peak-hour left-turn volume at this location is 70, which corresponds to the "50 or more" category in the table. For a two-lane rural highway intersection with three legs, the necessary peak-hour volume on the major highway is 50, which is less than the observed 391 shown in Figure 14. Therefore, a left-turn lane would be warranted at this location and should be considered.

Since the location is on the edge of suburban development, designers could choose to use criteria for suburban arterials, which are provided in Table 2 in Chapter 2. That table is reproduced here as Table 14 with the relevant volumes highlighted. For these conditions, with a peak-hour left-turn lane volume greater than 50 and a three-leg intersection, the necessary peak-hour volume on the major highway is 100, which is less than the observed volume of 391. Therefore, a left-turn lane would also be warranted at this location for suburban conditions.

Consideration of Access Management Guidelines

The state's *Access Management Manual* lists spacing criteria for state highway system routes that are not new highways on

Table 13. Recommended left-turn lane warrants for rural highways.

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

Table 14. Recommended left-turn lane warrants 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

new alignments. The relevant information is reproduced in Table 15, and it shows that, with a posted speed in excess of 50 mph, the spacing distance is 425 ft. A turn bay for another intersection or driveway is currently not present within that distance.

Design Result

A driveway is present approximately 300 ft west of the intersection. The design team decided to move forward with the turn lane at the intersection and to inform the property owner that a median opening at the driveway would not be considered when the highway is widened.

Design Application #2: Installation of an Exclusive Left-Turn Lane at an Unsignalized Intersection Between a Local Street and a State Highway

Context

Laurel Drive is a two-way, east-west local street, approximately 800 ft long, which provides access to several abutting properties including a church and several parcels accommodating office/light-industrial uses. The east end of Laurel Drive terminates at a cul-de-sac that accommodates separate driveways serving two properties. To the west, Laurel Drive terminates at an unsignalized T-intersection with Route 206. The Laurel Drive approach is stop controlled. Route 206 is a key north-south state highway through the area that widens immediately south of Laurel Drive to accommodate two continuous through lanes in the northbound direction and one continuous through lane in the southbound direction (see Figure 15). The posted speed on Route 206 is 45 mph. The intersection is located in a developing, suburban area that includes many large undeveloped parcels. Properties along the west side of Route 206 in the vicinity of Laurel Drive are currently undeveloped.

Table 15. Minimum spacing criteria for access connections.

Posted Speed (mph)	Distance (ft)
≤ 30	200
35	250
40	305
45	360
≥ 50	425

Design Request

The Mount Olive Township Police Department contacted the New Jersey Department of Transportation (NJDOT) with respect to the unsignalized intersection of Laurel Drive/Route 206, noting it as a safety concern due to a recent serious crash as well as many near misses. The intersection was noted as accommodating relatively high traffic volumes associated with employees and trucks traveling through the intersection to and from the abutting office/industrial uses. Representatives from the police department suggested NJDOT consider potential measures to improve the safety performance at this intersection. Improvements to the intersection were supported in writing by a signed petition from more than 130 employees from one of the tenants on a property abutting Laurel Drive.

Subsequent investigations by NJDOT staff revealed that there were nine crashes at the intersection in the most recent 3-year period for which data were available. Of these nine crashes, at least five appeared to be of a type that could potentially be mitigated by installation of an exclusive left-turn lane in the southbound direction. In addition, investigations by NJDOT indicated that installing an exclusive southbound left-turn lane would not interfere with the traffic flow on Route 206, yet would provide refuge for southbound vehicles turning left from Route 206 onto Laurel Drive.

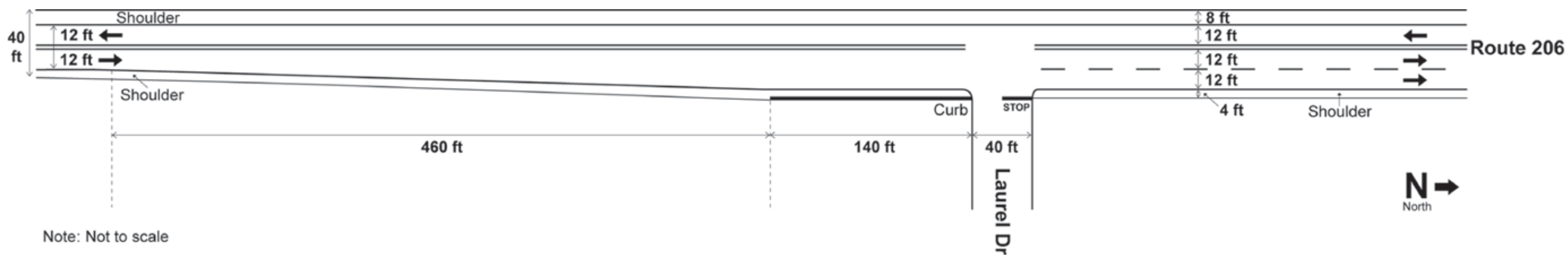


Figure 15. Laurel Drive/Route 206 intersection: existing condition.

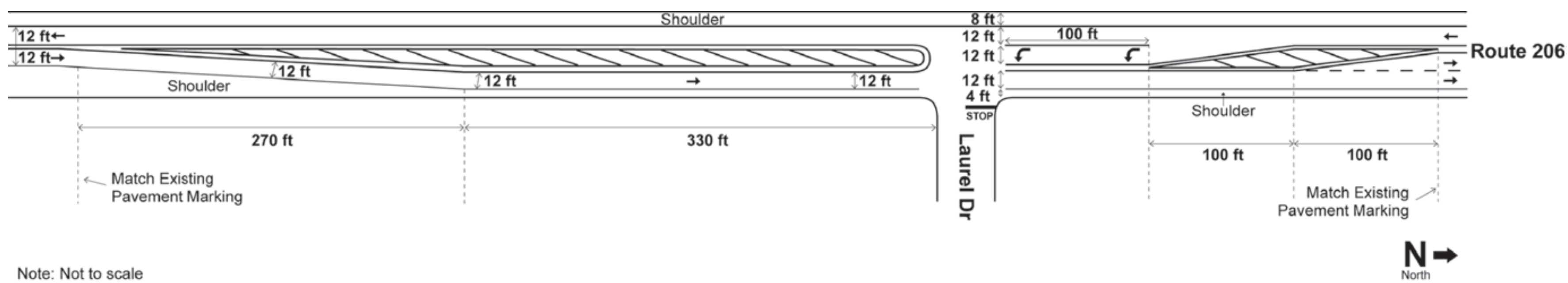


Figure 16. Laurel Drive/Route 206 intersection: proposed condition.

Design Considerations and Analysis

The primary design considerations evaluated by NJDOT included the existing traffic operations and safety characteristics of the intersection. In this particular instance, right-of-way and budgetary limitations eliminated the prospect of additional widening on Route 206, such that the addition of the left-turn lane needed to be accommodated within the existing pavement width along Route 206. Fortunately, despite these constraints, the intersection is located at a point where Route 206 widens from one lane to two lanes in the northbound direction. Therefore, the southbound left-turn lane on Route 206 was designed to be accommodated on existing pavement that formed the westerly travel lane, north of the intersection. Specifically, NJDOT addressed the following items as part of the design:

- **Determine the length of left-turn queue storage.** Multiple field visits to the site during various times determined that a 100-ft storage length for the left-turn lane would be adequate to accommodate expected vehicle queues. NJDOT also solicited input from the police department in determining the length of the left-turn storage lane. The police department concurred with the 100-ft length.
- **Determine the length of the shifting taper.** The length of the shifting taper on the upstream approach ("L") was calculated by NJDOT as:

  $L = WS/2$

  where W = transition width (12 ft) and S = speed in mph (45 mph).
  Consequently, $L = 270$ ft.
- **Determine the length of the downstream taper.** The downstream taper is intended to transition the cross section from one to two northbound lanes, north of Laurel Drive, beyond the left-turn lane. A downstream taper length of 100 ft—the MUTCD minimum for urban areas—was used by NJDOT.
- **Determine cross-sectional lane widths.** According to the New Jersey design manual, it is preferable to provide 12-ft standard lanes where possible. Because there was sufficient pavement width to accommodate 12-ft standard lane widths, the decision was made to apply the standard lane widths.

Design Result

Figure 16 illustrates the final design configuration with implementation of the exclusive southbound left-turn lane on Route 206. This improvement is planned for implementation in the near future.

Design Application #3: Addition of a Left-Turn Lane as Part of a "Road Diet" Treatment (Conversion of a Four-Lane Cross Section to a Three-Lane Cross Section)

Context

Titus Avenue is a four-lane east-west roadway approximately 3.2 miles long, located north of the City of Rochester, New York. Titus Avenue is classified as a minor arterial, has a posted speed of 35 mph, and is abutted by a mix of suburban commercial and residential development. Multilane county highways like Titus Avenue, due for major maintenance or reconstruction involving resurfacing and restriping, are periodically evaluated by Monroe County engineering staff to see if a multilane conversion (road diet) would be appropriate. This conversion involves changing the cross section of multilane highways to accommodate one travel lane in each direction, and converting the excess pavement width into auxiliary lanes (e.g., left-turn lanes, right-turn lanes, and/or shoulders) to realize the safety and traffic-calming benefits of such features. As part of the conversion project along Titus Avenue, a left-turn lane was provided on the westbound Titus Avenue approach at the unsignalized intersection with Seneca Avenue.

Design Request

As part of the Monroe County road diet program, engineering staff considered converting Titus Avenue from a four-lane cross section to a three-lane cross section. The cross section along Titus Avenue accommodated four 11-ft travel lanes, two in each direction. The cross section of Titus Avenue at the unsignalized intersection with Seneca Avenue is shown in Figure 17. Seneca Avenue is a major north-south collector roadway that serves a predominately residential neighborhood and terminates to the north at Titus Avenue. The posted speed on Seneca Avenue is 35 mph.

Design Considerations and Analysis

In general, the primary design considerations that were evaluated by Monroe County engineering staff included:

- Traffic volume,
- Safety,
- Signalized intersection capacity,
- Signal spacing, and
- Potential future roadside development.

(Source: Pictometry International, 2001
Markup credit: Monroe County Department of Transportation, New York, 2010.
Used by permission.)

Figure 17. Titus Avenue/Seneca Avenue intersection: previous four-lane cross section.

Specifically, Monroe County considers roadways with the following characteristics to be good candidates for road diets:

- Four travel lanes with peak directional traffic volumes that could be accommodated by one travel lane in each direction,
- A record of crash patterns correctable by the implementation of left-turn lanes,
- Signalized intersections that would maintain adequate capacity following the road diet conversion,
- No closely spaced signals, and
- Low potential for future roadside development.

With respect to the characteristics above, Titus Avenue was considered for a road diet based on its cross section (four travel lanes) and its peak directional traffic volume of approximately 260 vehicles per hour per lane (veh/hr/ln), which could be accommodated by one travel lane in each direction. This peak-hour directional volume is below the 450 veh/hr/ln threshold used by Monroe County for consideration in road diets. The road diet was implemented along a section of Titus Avenue away from closely spaced signalized intersections, and with low potential for future roadside development. The abutting properties were fully developed, and few land use changes were expected over time.

For most of the length of the road diet on Titus Avenue, the left-turn treatment consists of a center two-way left-turn lane (TWLTL) to accommodate vehicles turning left into the residences and suburban commercial establishments abutting the roadway. However, because Seneca Avenue is a key collector-level roadway that intersects Titus Avenue only from the south, an exclusive left-turn lane was striped on the westbound approach to accommodate left turns onto Titus Avenue.

Design Result

The final, improved cross section on Titus Avenue accommodates one 11-ft travel lane in each direction, one 12-ft center TWLTL, and 5-ft shoulders on both sides of the road. Figure 18 illustrates the final, improved configuration of the unsignalized Titus Avenue/Seneca Avenue intersection, with implementation of the three-lane road diet.

In 2009, a peak-hour directional volume of 315 veh/hr/ln was counted, suggesting that traffic volumes have continued to remain at a level easily accommodated by one travel lane in each direction. In addition, reportable crash rates were found to have decreased when comparing the 2 years (2002 to 2003) just prior to the road diet, to the 3 years (2005 to 2007) following the road diet.

Design Application #4: Installation of Left-Turn Lanes at an Unsignalized Rural Intersection Between a State Highway and a Local Road

[*Note: The intersection location in this design application has been made anonymous at the California Department of Transportation's (Caltrans') request. Street names have been changed.*]

(Source: Pictometry International, 2009
Markup credit: Monroe County Department of Transportation, New York, 2010.
Used by permission.)

Figure 18. Titus Avenue/Seneca Avenue intersection: final three-lane cross section.

Context

Franklin Avenue is a two-way, east-west local road located in a predominately rural area in central California. It accommodates one 12-ft travel lane in each direction, with no paved shoulder, and has a posted speed of 55 mph. Franklin Avenue intersects State Route (SR) 47 at an approximate 90-degree angle forming a four-legged, unsignalized intersection. SR 47 is a two-way, north-south highway that extends from the Pacific coast through central California. In the vicinity of Franklin Avenue, SR 47 has a posted speed of 55 mph and accommodates one 12-ft travel lane and an 8-ft shoulder in each direction.

At the Franklin Avenue/SR 47 intersection, only the eastbound and westbound (Franklin Avenue) approaches are stop controlled. No exclusive turn lanes are provided along either roadway. All four quadrants of the intersection are used for agricultural purposes, though a major casino/hotel complex is located approximately 1,600 ft east of the intersection on the south side of Franklin Avenue. Figure 19 illustrates pre-project conditions at the Franklin Avenue/SR 47 intersection.

Design Request

An investigation of this intersection was triggered by the Caltrans monitoring system, which reported this intersection as a location with a high concentration of crashes. The crash history at this intersection over the 36-month time period between April 1, 1998, and March 31, 2001, revealed six crashes that involved motorists passing stopped vehicles that were waiting to make left turns onto Franklin Avenue.

(Source: Caltrans. Used by permission.)

Figure 19. Franklin Avenue/SR 47 intersection: pre-project configuration.

Design Considerations and Analysis

A detailed analysis was initiated by Caltrans' Traffic Investigations Unit that included a safety review. After analyzing the crash patterns and conducting a field review, the unit determined that left-turn lanes should be installed on the northbound and southbound (SR 47) approaches to help improve safety at the intersection.

Caltrans addressed the following items as part of the design of the left-turn lanes.

Determine the Approach Taper Length

Section 405.2(2)(b) of the Caltrans *Highway Design Manual* (*29*) provides the following design guidance for calculating the length of the approach taper on highways with speeds of 45 mph or more:

$$L = WV$$

where:
L = length (ft)
W = lateral offset (ft)
V = speed (mph)

The Franklin Avenue/SR 47 intersection has a posted speed of 55 mph, and a lateral offset of 12 ft was desired. Therefore, the approach taper length for the left-turn lanes on SR 47 was calculated as follows:

$$\mathrm{L} = 12 \times 55 = 660 \text{ ft}$$

Determine the Bay Taper Length

Section 405.2(2)(c) of the Caltrans *Highway Design Manual* (*29*) provides the following design guidance for calculating the length of the bay taper:

> In urban areas, (bay taper) lengths of 60 ft and 90 ft are normally used. . . . On rural high-speed highways, a 120-ft length is considered appropriate (for the bay taper).

Because the intersection with Franklin Avenue is located along a rural, high-speed section of SR 47, a 120-ft bay taper was used.

Determine the Deceleration Length

Section 405.2(2)(d) of the Caltrans *Highway Design Manual* (*29*) provides the following design guidance for calculating the deceleration length:

> Deceleration Lane Length—Design speed of the roadway approaching the intersection should be the basis for determining

> deceleration lane length. It is desirable that deceleration take place entirely off the through traffic lanes. Deceleration lane lengths are given in Table 405.2B; the bay taper length is included. Where partial deceleration is permitted on the through lanes . . . design speeds in Table 405.2B may be reduced 10 mph to 20 mph for a lower entry speed.

Table 405.2B of the Caltrans *Highway Design Manual* is reproduced in this guide as Table 16.

Partial deceleration was used (as permitted by the Caltrans *Highway Design Manual*) with a speed reduction of 10 mph from the posted speed limit of 55 mph (i.e., 45 mph). Interpolation from Table 405.2B of the *Highway Design Manual* resulted in a total deceleration length of 375 ft.

Determine the Queue Storage Length

Section 405.2(2)(e) of the Caltrans *Highway Design Manual* (*29*) provides the following design guidance for calculating the queue storage length at unsignalized intersections:

> At unsignalized intersections, storage length may be based on the number of turning vehicles likely to arrive in an average 2-minute period during the peak hour. As a minimum, space for 2-passenger cars should be provided at 25 ft per car. If the peak hour truck traffic is 10 percent or more, space for one passenger car and one truck should be provided.

Because this highway segment accommodates 16 percent truck traffic, one truck length (i.e., 75 ft) plus one passenger car length (i.e., 25 ft) were used to calculate the 100-ft storage distance.

Determine Left-Turn Lane Width

A 12-ft lane was used, as per the Caltrans *Highway Design Manual* (*29*), Section 405.22a, which states that the lane width for both single and double left-turn lanes on state highways shall be 12 ft.

Design Result

Figure 20 illustrates the final design configuration of the Franklin Avenue/SR 47 intersection with implementation of northbound and southbound left-turn lanes on SR 47. Right-of-way was acquired from parcels on the northeast, southeast, and southwest corners to accommodate the widening.

Table 16. Deceleration lane length for left-turn lanes (*29*).

Design Speed (mph)	Length to Stop (ft)
30	235
40	315
50	435
60	530

(Source: Caltrans. Used by permission.)

Figure 20. Franklin Avenue/SR 47 intersection: post-project configuration.

Post-project crash data for the time period between July 1, 2007, and December 31, 2008, suggest that the previous pattern of passing/left-turn collisions was mitigated by installation of the left-turn lanes on SR 47.

Design Application #5: Installation of Unsignalized "J-Turn" Intersections Along a State Highway

Context

US 15 is a two-way, rural principal arterial following a north-south alignment across Maryland from the Pennsylvania border to the Virginia border. US 15 is not only a heavily used commuter route for local and regional traffic, but also serves as a major north-south highway route on the East Coast, accommodating traffic from New York State to South Carolina. Access control along the entire interstate length of US 15 ranges from fully access-controlled freeway segments with grade-separated interchanges, to partially access-controlled expressway segments with at-grade intersections.

The subject section of US 15, located between MD 26 and the Pennsylvania border in Frederick County, is a four-lane divided highway with two 12-ft lanes in each direction, plus a 4-ft median shoulder and a 10-ft right shoulder. The directional roadways are separated by a wide grass median that varies in width from 51 ft to 72 ft. Full-access at-grade median breaks are provided at intersections with major public roads. These intersections are generally spaced several thousand feet apart, and have separate exclusive left-turn and right-turn lanes on US 15. The speed limit in this section of US 15 is 55 mph, with 85th percentile speeds in the 60 to 65 mph range. Trucks account for approximately 15 percent of the vehicle mix in the traffic stream.

Design Request

In the 1970s and 1980s, Frederick County began to experience significant residential and commercial growth, which continues today, making it one of the fastest growing counties in Maryland. During the 1980s and 1990s, average daily traffic (ADT) volumes along US 15 were observed to increase at a rate of 7 percent annually. Traffic volumes—particularly left turns to and from US 15 and the intersecting side streets—also increased at most intersections, increasing the pressure for traffic signal installations. These turns also generated an increase in angle-type crashes. In addition, the transitions in design elements between the freeway and expressway segments along US 15 were a safety concern to the Maryland State Highway Administration (MDSHA) because these changes tend to violate driver expectancy and lessen driver awareness and responsiveness. Although there are long-range plans to convert US 15 into a freeway with fully controlled access, interchanges, and service roads, MDSHA had no immediate plans to provide such improvements along the subject portion of US 15.

MDSHA also developed the position, with the full cooperation and support of Frederick County government, that traffic signals would not be placed at any location along the subject section of US 15, even where warranted by the *Manual on Uniform Traffic Control Devices.* MDSHA was concerned, based on past experiences at other similar locations, that the mixing of freeway and expressway design elements, high traffic volumes, and high speeds would create overly hazardous conditions with the potential for serious rear-end and angle crashes. These concerns, coupled with the rapid traffic growth and increasing crash trends along the subject segment of US 15, led MDSHA to consider application of the "directional crossover" concept—also known as a "J-turn" or "superstreet."

Design Considerations and Analysis

A variety of other potential design treatment options were considered to address the safety and operational issues along the subject segment of US 15, including the following:

- **Installation of traffic signals** was not selected due to concerns that the crash experience, particularly in terms of severity, would likely increase.
- **Complete crossover closure** was not selected because it was MDSHA's intent to only (initially) prohibit the problem movement (i.e., left turns), while still allowing a high level of access to and from the side streets.
- **Installation of jug handles** was ruled out because the jug handle intersection treatment would not eliminate through and left-turn movements from the side streets, and would require signalization.

Ultimately, MDSHA selected the J-turn as the preferred treatment along the subject segment of US 15. Figure 21 is an aerial photograph of an existing (signalized) J-turn location in Troy, Michigan, illustrating the typical geometric configuration of the J-turn treatment (*30*).

The primary purpose of the J-turn treatment is to eliminate the left turns and through movements from the side street. As shown in Figure 21, this is accomplished by installing a raised channelizing island within the median crossover at the subject intersection as well as channelizing islands on the side-street approaches, such that only right turns to and from the side street and left turns from the major street to the side street are allowed. Motorists desiring to make left turns and through movements from the side-street approaches must make these movements indirectly, via a right turn from the side street, followed by a U-turn at a median break located downstream of the subject intersection. The J-turn configuration can negate the need for signalization at the subject intersection, eliminates the cause of most angle collisions (i.e., left-turn and through movements), significantly reduces the number of conflict points, and reduces the potential for confusion among drivers turning within the median crossover.

The intersections of the following roads, located along the subject segment of US 15, were selected by MDSHA for the J-turn treatment:

- College Lane,
- Old Frederick Road,
- Sundays Lane,
- Biggs Ford Road,
- Willow Road,
- Monocacy Boulevard, and
- Hayward Road–MD 355.

(Source: Google Earth™ mapping service.)

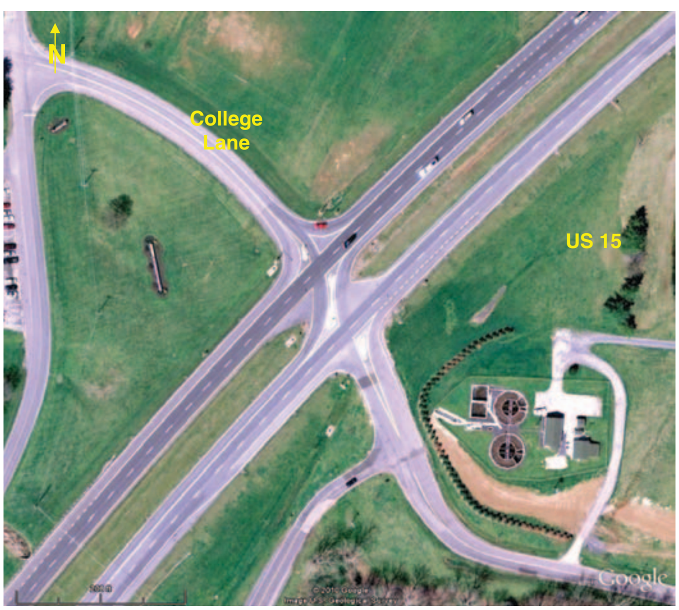
Figure 21. Aerial photograph of typical J-turn intersection treatment (30).

This design application focuses on the J-turn treatment at the intersection of College Lane and US 15. In the vicinity of College Lane, US 15 bisects the campus of Mount Saint Mary's University, just south of Emmitsburg, Maryland. College Lane serves as the primary access to the main campus located west of US 15, and to Knott Arena and the athletic complexes located east of US 15. In addition to traffic volumes at the intersection that nearly met traffic signal warrants under normal weekday conditions, Knott Arena generated high volumes of traffic on nights and weekends as a result of sporting/special events. These volumes became so high that university officials manually directed traffic at the intersection. Working closely with the university, MDSHA constructed a J-turn intersection treatment at the intersection in August 1994. The treatment included two U-turn crossovers on US 15, located approximately 2,000 ft south and 3,000 ft north of College Lane.

Design Result

Figure 22 illustrates the final design configuration at the College Lane/US 15 intersection with implementation of the J-turn treatment.

A before study of the College Lane/US 15 intersection's crash history prior to the J-turn improvement revealed that 11 crashes occurred during the 3-year period between August 1991 and July 1994. These crashes included nine angle crashes, 8 injury crashes, and 19 personal injuries. During the 3-year after study period from September 1994 through August 1997, following construction of the J-turn, there was only one reported crash, which was a left-turn collision that resulted in two injuries.



(Source: Google Earth™ mapping service.)

Figure 22. College Lane/US 15 intersection: final J-turn configuration.

Furthermore, there were no reported crashes during the after period at the U-turn crossovers located on US 15 north and south of College Lane.

MDSHA's experiences with the J-turn intersection treatment have revealed several important design considerations:

- **Provide adequate sight distance for major-street left turns.** The proper alignment of the major-street left-turn lanes within the median is crucial to maintaining adequate sight distance for drivers making these turns. Vehicles making left turns from the major street should be aligned such that drivers' sight lines to oncoming through traffic are not obstructed by vehicles queued in the opposing left-turn lane.
- **Provide "loons" for large U-turning vehicles at downstream median breaks.** It is important that vehicles with large turning radii (e.g., tractor-trailers) are able to complete the U-turn maneuver at the downstream median breaks without tracking beyond the edge of pavement. To accommodate these large-radius turns, particularly along corridors with narrow medians, extra widening of the pavement beyond the normal shoulder (i.e., "loons") should be installed to accommodate such vehicles.
- **Provide adequate distance to the downstream median break.** As noted previously, drivers desiring to make left turns and through movements from the side-street approaches must make these movements indirectly, via a right turn from the side street, followed by a U-turn at a median break located downstream of the subject intersection. These drivers must be provided with enough distance between the intersection and the downstream U-turn median break to safely merge with, and across, major-street traffic in order to turn into the median break. However, as traffic volumes on the major street increase over time, and the number of available gaps in the traffic stream decreases, this maneuver can become more difficult. Therefore, it is important that future traffic growth along the major street in both directions be considered in the design. MDSHA's experience has shown that a separation distance of 1,700 ft was sufficient for roadways with average annual daily traffic (AADT) volumes of approximately 20,000 vehicles per day. However, as the AADT on the same roadway increased to approximately 43,000 vehicles per day, a separation distance of 3,000 ft was desirable.

Design Application #6: Installation of a Left-Turn Passing Blister at an Unsignalized Intersection Between a Local Street and a State Highway

(*Note: The intersection location in this design application has been made anonymous at the request of the contributing agency. Street names have been changed.*)

Context

Sherwood Road is a two-way, east-west local street that provides access to a rapidly developing industrial park area that currently includes a furniture store and a major distribution center for a fast-food restaurant chain. The 1997 AADT on Sherwood Road was 1,950 vehicles per day, but daily and peak-hour volumes are expected to increase significantly in the future as a result of intensifying operations for the existing industrial uses along Sherwood Road, and anticipated new developments in the industrial park. Sherwood Road has a posted speed of 30 mph and is aligned within an existing right-of-way of approximately 40 ft. Sherwood Road terminates to the west at an approximate 65-degree T-intersection with SR 46.

State Road 46 (SR 46) is a two-way, north-south state highway with a 1997 AADT of 8,460 vehicles per day and a posted speed of 50 mph. SR 46 is classified as a rural principal arterial and is the primary north-south highway through the region. In the vicinity of Sherwood Road, SR 46 has an approximate 60-ft right-of-way.

The Sherwood Road/SR 46 intersection is unsignalized, and only the Sherwood Road (westbound) approach is stop controlled. Both roadways accommodate one 12-ft travel lane in each direction. No turn lanes or shoulders have been constructed along either roadway. There is adequate intersection sight distance in both directions.

Design Request

Both the local chamber of commerce and the local police department contacted the state department of transportation to request that the unsignalized Sherwood Road/SR 46 intersection be investigated for possible improvements. In the correspondence, they noted the large amount of recent—and anticipated future—traffic growth at the intersection associated with the expanding industrial park. In addition, they noted that the increasing traffic volumes are exacerbating safety issues, resulting in several crashes and near misses, including one crash that resulted in a fatality.

Design Considerations and Analysis

In response to these local requests, a comprehensive and detailed investigation of traffic operations and safety was conducted by state transportation engineering staff. The study found high peak-hour volumes of passenger vehicles during time periods of worker shift changes, as well as a relatively high percentage of trucks traveling through the intersection. Although the number of crashes occurring during the most recently available 3-year period was not particularly high, the fatality indicated that the intersection should receive priority treatment.

The design considerations evaluated by staff included the existing and projected future traffic operations and safety characteristics of the intersection. The following is a listing of the primary improvements staff recommended for implementation at the Sherwood Road/SR 46 intersection, based on their analysis findings:

- Construct a left-turn passing blister on the west side of SR 46 in the vicinity of Sherwood Road to allow southbound through vehicles to bypass vehicles making a southbound left turn onto Sherwood Road.
- Construct a northbound right-turn lane.
- Widen the Sherwood Road (westbound) approach to accommodate separate exclusive left-turn and right-turn lanes at the intersection.
- Construct shoulders on both SR 46 and Sherwood Road.

Staff determined that additional right-of-way was needed to accommodate the improvements above. The width of the additional right-of-way acquisition varied from approximately 20 to 30 ft.

The state's design manual provides guidelines for various left-turn treatments (i.e., left-turn lanes, left-turn passing blisters, etc.). The following sections were referenced as part of the design of the left-turn passing blister:

- **Determine the type of left-turn treatment on the southbound approach of SR 46.** As stated in the design manual:

> At some three-legged intersections, it may be desirable to provide a passing blister to relieve congestion due to left-turning vehicles . . . Passing blisters may be provided at the intersection of all public roads and streets on 2-lane State highways with a design year ADT of 5000 or greater . . . The decision on whether to use either a channelized left-turn lane or a passing blister will be based on accident history, right-of-way availability, through and turning traffic volumes, design speed and available sight distance. A channelized left-turn lane should be provided if the left-turn volumes are high enough that a left-turn lane is warranted . . .

- **Determine the length of the passing blister and associated tapers.** The length of the passing blister, and the associated upstream and downstream tapers, are based on speed as described in the state's manual, shown here as Table 17. Because the posted speed on SR 46 is 50 mph, the minimum length of the passing blister was determined to be 197 ft. Staff selected a length of 328 ft for use in the design. The minimum length of both tapers was determined to be 295 ft (see Table 17) and these lengths were incorporated into the design.

Design Result

Figure 23 illustrates the final design configuration of the Sherwood Road/SR 46 intersection with implementation of

Table 17. Minimum dimensions for passing blisters on two-lane highways for design application #5.

Design Speed, v_D (mph)	T_1 (ft)	L (ft)	T_2 (ft)
≤ 31	148	148	148
31 < v_D < 50	197	148	197
≥ 50	295	197	295

NOTES:
- "T_1" and "T_2" are the minimum lengths of the upstream and downstream tapers, respectively; and
- "L" is the minimum length of the passing blister.

the southbound left-turn passing blister on SR 46 and other improvements. Construction of this improvement was completed in 2005.

Design Application #7: Installation of Left-Turn Lanes at an Unsignalized Rural Intersection Between a State Highway and a County Road

(*Note: The intersection location in this design application has been made anonymous at the request of the contributing agency. Street names have been changed.*)

Context

County Road 800 North (CR 800N) is a two-way, east-west county highway through a rural area. CR 800N is classified as a rural minor collector and accommodates one 11-ft lane, plus a 6-ft shoulder, in each direction. The 1997 AADT on CR 800N was 1,040 vehicles per day, of which approximately 10 percent were commercial vehicles. CR 800N has a posted speed of 45 mph and intersects US 281 (a state highway) at an approximate 90-degree angle, forming a four-legged intersection. The existing right-of-way along CR 800N is 33 ft.

US 281 is a two-way highway that extends north-south across nearly the entire state. It is classified as a rural princi-

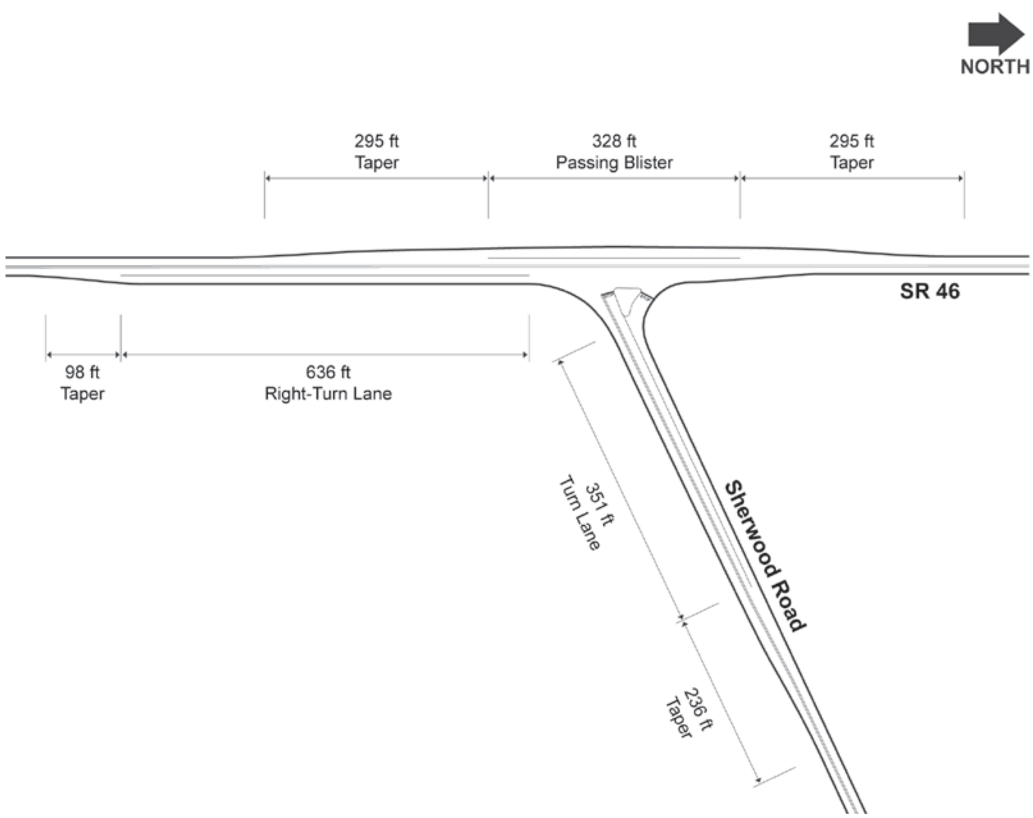


Figure 23. Sherwood Road/SR 46 intersection: post-implementation condition.

pal arterial and had a 1997 AADT of 8,400 vehicles per day, of which approximately 13 percent were commercial vehicles. US 281 has a posted speed of 55 mph and accommodates one 12-ft travel lane, plus a 4-ft shoulder, in each direction. From a regional perspective, US 281 serves as a major north-south highway route through the state. In the vicinity of CR 800N, US 281 has right-of-way that varies from 40 to 70 ft.

The US 281/CR 800N intersection is unsignalized, and only the CR 800N (eastbound and westbound) approaches are stop controlled. No turn lanes are provided along either roadway. All four quadrants of the intersection are used for agricultural purposes, although development in the area is expanding, and the southwest quadrant was subdivided for residential housing.

Design Request

The need for improvements at the US 281/CR 800N intersection was reported to state department of transportation staff in a district office by local citizens who noted several crashes in the northbound direction and increased traffic volumes associated with development in the area.

Design Considerations and Analysis

An engineering investigation of traffic operations and safety at the US 281/CR 800N intersection was conducted by state transportation engineering staff in the district office. A review of crash data for the 4-year period from 1993 through 1996 revealed that of the eight crashes that occurred during this time, four (50 percent) were rear-end crashes. These rear-end crashes resulted in a total of five injuries to vehicle occupants. In addition, the state's investigation revealed delays for northbound and southbound through traffic when northbound and southbound left-turning vehicles were present and waiting to turn. Because the need for an improvement was particularly acute in the northbound direction, staff initially recommended that a northbound "passing blister" (i.e., pavement widening to accommodate through traffic traveling to the right of left-turning vehicles) be installed. However, based on a field visit by the design team (i.e., including district staff and consultants), it was agreed that opposing left-turn lanes on both the northbound and southbound approaches would provide improved intersection geometrics and a more smooth lane transition for through traffic for only a small incremental increase in cost. Based on the findings of this investigation, state staff recommended implementing left-turn lanes in both the northbound and southbound directions on US 281 at its intersection with CR 800N. Staff determined that additional right-of-way was needed along US 281—as well as a small amount along CR 800N—to accommodate the left-turn lanes.

The state's design manual provides warrants for determining whether an exclusive left-turn lane is needed. In addition, the state design manual provides the following guidance for determining the lengths for deceleration (L_D), taper (L_T), and queue storage (L_S) for left-turn lanes:

> The length of auxiliary lanes will be determined by some combination of its taper length (L_T), deceleration length (L_D) and storage length (L_S) and by the mainline functional classification. Length considerations for the various classifications are as follows:

Classification	**Functional Length**
Rural arterials	$L_T + L_D + L_S$
Urban arterials and other facilities	$L_T + L_D + L_S$ (Desirable)
Stop or t facilities	$L_T + L_S$

> NOTE:
> L_T = length of taper (100 ft or more)
> L_D = length of deceleration (only a consideration at free-flowing legs of stop-controlled intersections, at signalized intersections, and at free-flowing turning roadways with turn lanes)
> L_S = length of storage

The following will apply.

1. *Taper.* For tangent approaches, the department's practice is to use a 100-ft straight-line taper at the beginning of a single turn lane. . . .
2. *Deceleration.* For rural facilities, the deceleration distance (LD) should meet the criteria presented in Table 18. In addition, the values determined from Table 18 should be adjusted for grades. Table 18 also provides these grade adjustment factors. These distances are desirable on urban facilities; however, this is not always feasible. Under restricted urban conditions, deceleration may have to be accomplished entirely within the travel lane. For these cases, the length of turn lane will be determined solely on the basis of providing adequate vehicle storage (i.e., $L_D = 0.0$ m).
3. *Storage Length (Signalized Intersections) [DOES NOT APPLY]*
4. *Storage Length (Unsignalized Intersections).* The storage length should be sufficient to avoid the possibility of left-turning vehicles stopping in the through lanes and waiting for a gap in the opposing traffic flow. The minimum storage length should have sufficient length to accommodate the expected number of turning vehicles likely to arrive in an average 2-minute period within the design hour. At a minimum, space should be provided for two passenger cars. If truck traffic exceeds 10 percent, space should be provided for at least one passenger car and one truck. The recommended storage lengths for right- and left-turn lanes at unsignalized intersections are provided in Table 19.

Based on the guidance for taper, a left-turn taper of 100 ft was provided on both the northbound and southbound approaches on US 281. In order to avoid an excessive project

Table 18. Deceleration distances for turning lanes.

Design Speed (mph)	Desirable L_D Full-Width Auxiliary Lane (ft)
70	944
65	850
60	756
55	662
50	567
45	473
40	379
35	284

Grade Adjustment Factors				
Downgrade				
6.00 to 5.00%	4.99 to 4.00%	3.99 to 3.00%	2.99 to 2.01%	2.00 to 0%
1.35	1.28	1.20	1.10	1.10
Upgrade				
0 to 2.00 %	2.01 to 2.99%	3.00 to 3.99%	4.00 to 4.99%	5.00 to 6.00%
1.00	0.95	0.90	0.85	0.80

NOTE: Multiplying the length L_D by the grade adjustment factor will give the deceleration lane length adjusted for grade. Adjustment factors apply to all design speeds.

length, deceleration lengths were not planned for the left-turn lanes due to the relatively low volume of left-turn movements and the available sight distance along this section of US 281. The storage lengths for the northbound and southbound left-turn lanes (i.e., approximately 98 ft) were identified based on projected future design volumes of northbound and south-bound left-turning traffic.

Design Result

Figure 24 illustrates the final design configuration of the US 281/CR 800N intersection with implementation of north-bound and southbound left-turn lanes on US 281.

Design Application #8: Follow-Up Traffic Studies to Verify the Need for an Unsignalized Left-Turn Lane in Conjunction with a Proposed Development

Context

SR 542 is a two-way east-west state highway located in central Florida. Within the city of Winter Haven, SR 542 is also known as Dundee Road and is classified as an arterial. SR 542 accommodates one travel lane in each direction, has a posted speed of 55 mph, and had an ADT of 16,900 vehicles per day (vpd) in 2004.

Table 19. Recommended storage lengths (L_S) for unsignalized intersections.

Turning DHV (veh/hr)	L_S (ft)
< 60	50–80
61–120	100
121–180	150
>180	200 or greater

NOTES:
- "DHV" is the design hourly volume.
- "L_S" is the minimum storage length for the left-turn lane.

Design Request

In 2004, the Florida Department of Transportation (FDOT) received an access permit request associated with the proposed development of a 15,784-square-ft outpatient surgery center on the south side of SR 542, east of Florida Drive, in Winter Haven. As part of this development, two full-access, stop-controlled private driveways were proposed to connect to the south side of SR 542, approximately 300 ft east and 900 ft east of Florida Drive, respectively. The second driveway was proposed by the applicant as part of the initial application in anticipation of a second development phase occurring on the site as part of a planned future expansion.

Design Considerations and Analysis

As part of the access permitting process, FDOT's access management specialists reviewed the proposed development plans and concluded that widening of SR 542 would be required to accommodate a westbound left-turn lane at the westerly site-access driveway. FDOT reached this conclusion based on:

- A review of existing traffic volumes and projected future traffic growth along SR 542;
- Trip generation estimates for the proposed development, based on standard trip rates from ITE; and
- FDOT's assumed trip distribution characteristics (east/west split) for the proposed development.

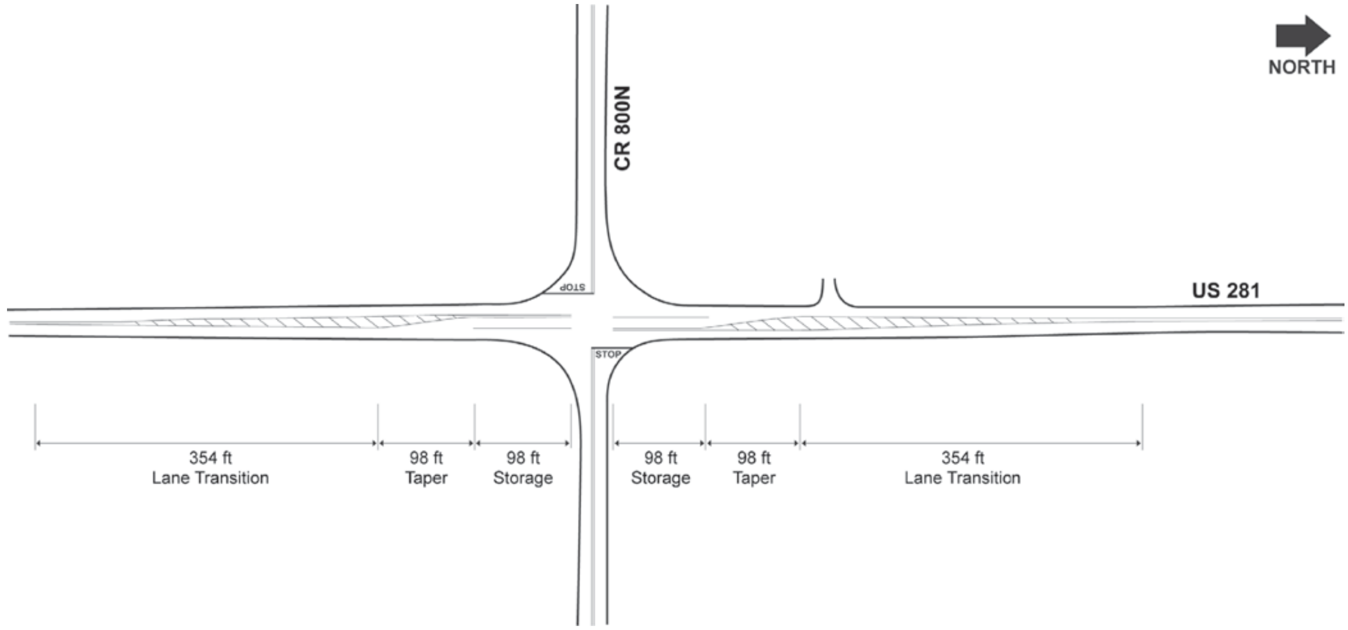


Figure 24. US 281/CR 800N intersection: post-improvement condition.

However, the applicant disputed the use of ITE trip generation rates, noting that the proposed surgery center would have unique operational characteristics, quite unlike the traditional medical/dental office uses reflected in the ITE database. In particular, the applicant noted that the surgery center would have:

- No patient examinations, check-ups, or follow-up appointments;
- No prescription refills;
- No visits by nurse practitioners or other assistants;
- An average patient stay of 4 hours (longer than that suggested by ITE); and
- Patients arriving for purposes of surgery only and leaving the facility by the early afternoon.

The applicant contended that these unique operational characteristics would result in vehicle trip generation at the proposed site that would be lower than that calculated using ITE rates, such that the threshold for installation of a left-turn lane on SR 542 would not be met.

In addition, there was a dispute about the trip distribution pattern assumed by the applicant. FDOT projected that more site-generated traffic would be oriented to and from the east, given the proximity of the site to US 27, a major north-south state highway located approximately 3.25 miles east of the site.

Design Result

Given the circumstances of this particular access permit application and uncertainties surrounding the operational aspects of the proposed use, FDOT and the applicant reached a compromise: FDOT agreed to grant a conditional access permit without requiring construction of the westbound left-turn lane, provided that the applicant supplied FDOT with a bond for the construction cost of the left-turn lane. Furthermore, after the proposed surgery center was open and operational for 1 month, the applicant would be obligated to conduct a follow-up traffic study—based on actual field conditions and mutually agreed-upon criteria—to reassess whether or not the left turn would be warranted. If the left-turn lane was found not to be warranted after 1 month, a second follow-up traffic study would be conducted 1 year after the opening of the surgery center for the same purpose. If either follow-up study determined that a left-turn lane was warranted, either the applicant would be obligated to construct the left-turn lane in conformance with FDOT standards, or FDOT would cash the bond and construct the left-turn lane. If the left-turn lane was found not to be warranted under either circumstance, FDOT would return the bond to the applicant.

For purposes of the follow-up traffic studies, the left-turn lane would be warranted if both of the following criteria were met:

- There were more than 10 actual left turns from SR 542 into the site during the typical weekday AM peak hour.
- There were less than 60 5-sec eastbound vehicle gaps (time between vehicles approaching from the west) during the typical weekday AM peak hour at the site driveway. For purposes of the study, a 15-sec vehicle gap would be recorded as three 5-sec gaps.

(Source: Google Earth™ mapping service.)

Figure 25. Subject development, located south of SR 542.

Figure 25 illustrates the post-development configuration of the facility's access driveways on the south side of SR 542.

Design Application #9: Installation of Left-Turn Lanes at Unsignalized Intersections in Conjunction with a Proposed Development

(*Note: The intersection location in this design application has been made anonymous at the request of the contributing agency. Street names have been changed.*)

Context

Route 52 is a two-way north-south state highway. Within the area of the proposed development in the town of Harrison, Route 52 has a posted speed of 35 mph and is classified as a minor rural arterial. The original cross section of Route 52 in Harrison accommodated three lanes—one lane northbound and two lanes—and had an ADT of 18,000 vpd north of Meadow Terrace in the year 2008.

Within Harrison, two local streets—Meadow Terrace and Parent Lane—intersect Route 52 as unsignalized T-intersections located within approximately 200 ft of each other. Meadow Terrace, the northerly roadway, intersects Route 52 from the west and provides access to approximately two dozen single-family homes and a few small commercial establishments before dead-ending approximately 1,600 ft west of Route 52. Parent Lane, the southerly roadway, intersects Route 52 from the east and serves as one of two access points to an elementary school, located approximately 1,400 ft east of Route 52. Both intersections are stop controlled on the side-street approaches to Route 52. This area of Harrison is primarily suburban with many single-family residences, as well as some supporting commercial uses primarily abutting Route 52.

Design Request

In 2004, the state received an application for the proposed development of an approximate 291,000-square-ft retail development on the west side of Route 52 in Harrison. The proposed development included a 133,000-square-ft home improvement store, a 63,000-square-ft supermarket with a 7,000-square-ft mezzanine, 74,000 square ft of retail space, and two free-standing restaurants totaling 14,000 square ft. Approximately 1,500 parking spaces would be provided as part of the development.

The proposed development was projected to generate 1,962 vehicle trips during the weekday PM peak hour and 2,648 vehicle trips during the Saturday midday peak hour. Access to the development was proposed via two signalized access driveways connecting to the west side of Route 52. Both driveways were proposed to be aligned opposite existing local streets intersecting Route 52 from the east. The southerly access driveway was proposed to be located on the west side of Route 52, approximately 300 ft north of Meadow Terrace.

Given the size of the proposed development relative to the available capacity of the street network in the surrounding area, a significant number of specific roadway improvements were proposed to accommodate the projected future travel demands associated with the development. These improvements included traffic signal installations at a number of stop-controlled intersections; roadway widening to accommodate exclusive right-turn, left-turn, and through lanes; realignment of existing roadways; widening of the off-ramps at a nearby interchange; installation of sidewalks; installation of guide signs; and other transportation-related improvements. Included in the package of improvements proposed by the applicant were provisions to widen Route 52 to accommodate back-to-back left-turn lanes at the unsignalized T-intersections of Meadow Terrace/Route 52 and Parent Lane/Route 52. Acquisition of right-of-way along Route 52 was not required to accommodate this improvement.

Design Considerations and Analysis

During the development review process, the applicant's engineer proposed a widening of Route 52 along the site frontage (north of Meadow Terrace) to provide additional lanes. This widening provided an opportunity to introduce back-to-back left-turn lanes on Route 52 at the adjacent intersections of Parent Lane and Meadow Terrace. The state's engineering staff felt that the left-turn lanes were necessary to remove left-turning motorists from the through travel lanes, thereby improving capacity, operations, and safety in the vicinity of the development.

As a result, the applicant included improvement plans for a northbound left-turn lane at the Meadow Terrace/Route 52 intersection, as well as a southbound left-turn lane at the Parent Lane/Route 52 intersection. The purpose of these left-turn lanes was to provide a safe refuge for left-turning vehicles waiting to turn across oncoming through traffic, and reduce the propensity for delays associated with northbound and southbound through traffic waiting behind left-turning vehicles at these intersections. The lengths of both left-turn lanes were limited by the existing separation distance between Meadow Terrace and Parent Lane (approximately 200 ft).

Design Result

Figure 26 illustrates the final design at the Meadow Terrace/Route 52 and Parent Place/Route 52 intersections. The design maintains two 11-ft travel lanes in the southbound direction and one 11-ft travel lane in the northbound direction, and provides back-to-back 11-ft exclusive left-turn lanes. A transition taper of approximately 75 ft separates the left-turn lanes. Approximately 75 ft of vehicle storage is provided in the southbound left-turn lane at Parent Lane, which is sufficient to accommodate three passenger cars, or one tractor-trailer and one passenger car. Approximately 50 ft of vehicle storage is provided in the northbound left-turn lane at Meadow Terrace, which is sufficient to accommodate two passenger cars or one tractor-trailer. In the northbound direction, the left-turn lane continues north of Meadow Terrace and becomes an exclusive left-turn lane at the adjacent signalized intersection providing access to the proposed development.

Design Application #10: Installation of an Exclusive Left-Turn Lane at an Unsignalized Suburban Intersection Between a State Highway and a Local Street

Context

SR 76 (also known as SW Kanner Highway) is a two-way state highway that extends between Lake Okeechobee and the city of Stuart in Florida. SR 76 is primarily an east-west highway for much of its length but generally follows a north-south alignment near the intersection with SW Tropical Avenue. SW Tropical Avenue is a two-way local street in Stuart that intersects the west side of SR 76, forming an unsignalized T-intersection. The intersection is stop controlled on the SW Tropical Avenue (eastbound) approach only. In the vicinity of SW Tropical Avenue, SR 76 is an undivided roadway classified as an arterial. SR 76 also has a posted speed of 50 mph, and initially accommodated one 12-ft lane and a 6-ft paved shoulder in each direction. SW Tropical Avenue has a posted speed of 30 mph and accommodates one 11-ft lane in each direction and no shoulder. The SR 76/SW Tropical Avenue

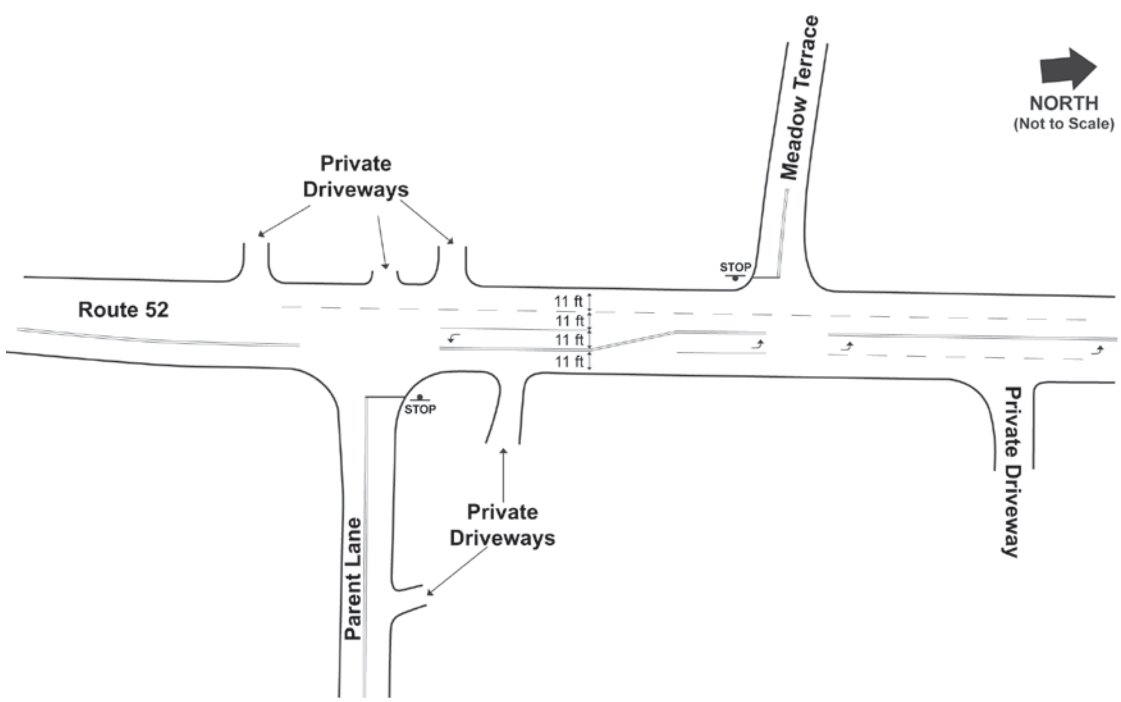


Figure 26. Final geometric layout of Meadow Terrace/Route 52 and Parent Lane/Route 52 intersections.

intersection is located in a primarily suburban area of Stuart. Land uses adjacent to the intersection include residential housing (single-family homes and mobile homes) to the west of SR 76, and a nursery and golf course to the east. Figure 27 illustrates the original, pre-improvement configuration of the SR 76/Tropical Avenue intersection.

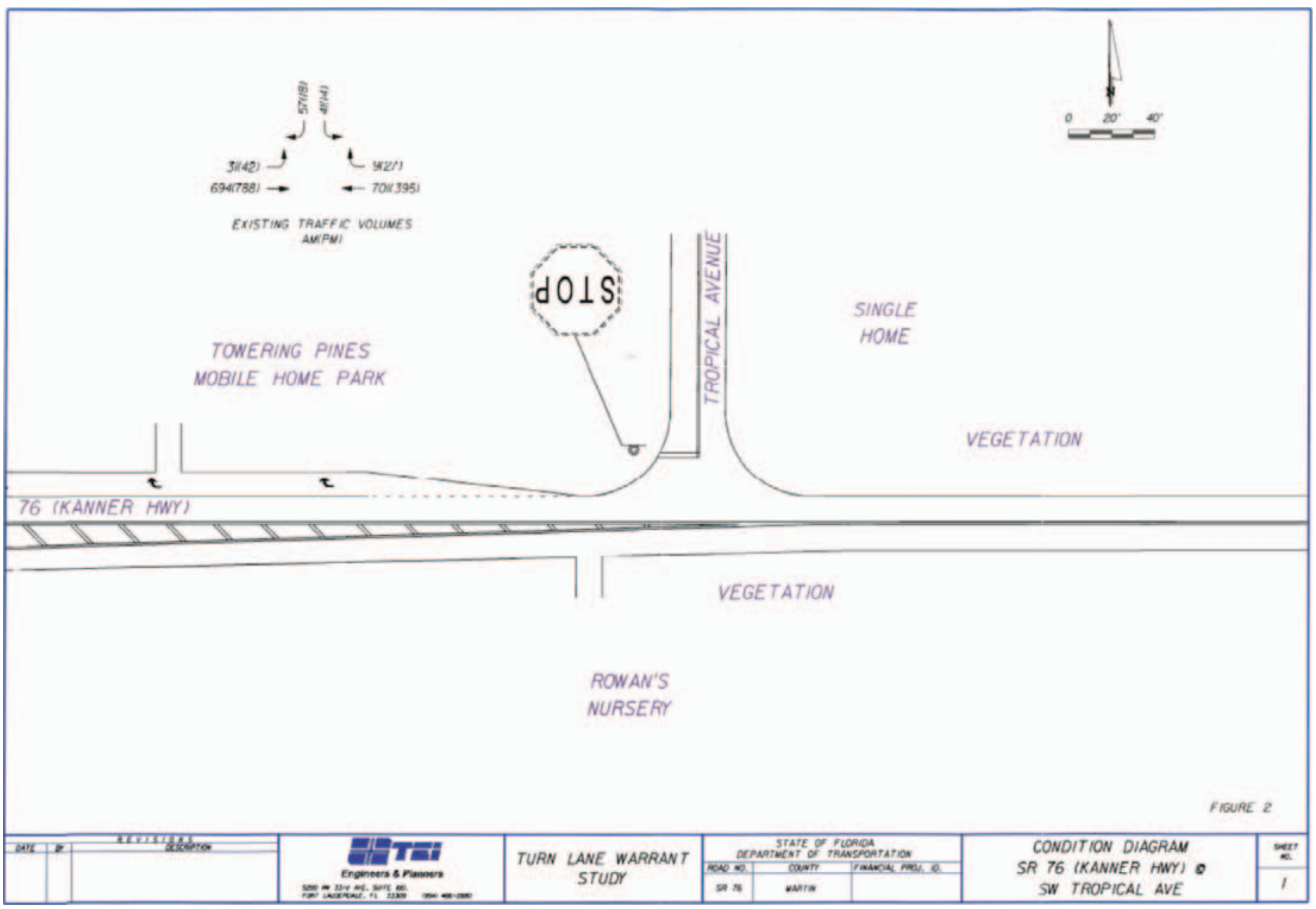
Design Request

In response to a citizen's letter requesting that a separate left-turn lane be provided on SR 76 to accommodate turns onto SW Tropical Avenue, District 4 staff at FDOT conducted an engineering study of traffic operations and safety at the SR 76/SW Tropical Avenue intersection. As part of this study, pedestrian counts and vehicular turning-movement counts were conducted at the intersection during the typical weekday morning (7:00 to 10:00 AM), midday (11:00 AM to 1:00 PM), and afternoon (3:00 to 6:00 PM) peak hours. In addition, crash data spanning the 3-year period from 1997 through 1999 were reviewed.

Design Considerations and Analysis

The crash data revealed that of the nine crashes that occurred during the 3-year study period, a majority (56 percent) were rear-end crashes that were attributed to the lack of either a northbound left-turn lane on SR 76 or a southbound right-turn lane. The contributing causes included "improper passing," "careless driving," and "failure to yield right-of-way." Over the 3-year period, crashes at the intersection resulted in three injuries to vehicle occupants.

Left-turn volumes from SR 76 onto SW Tropical Avenue were found to be 31 vehicles during the weekday morning peak hour, 18 vehicles during the weekday midday peak hour, and 42 vehicles during the weekday afternoon peak hours. Field observations revealed that motorists traveling at relatively high speeds on northbound SR 76 were forced to decelerate to a stop and wait for left-turning vehicles to turn, creating potential conflicts. Some northbound motorists on SR 76 were also observed using the shoulder to pass stopped left-turning vehicles on the right (east) side. These behaviors



(Source: Florida Department of Transportation. Used by permission.)

Figure 27. Pre-improvement condition diagram: SR 76/Tropical Avenue intersection.

resulted in damage to the east shoulder on SR 76 and created potential sideswipe conflicts with left-turning vehicles. A total of five pedestrians were observed during the course of the 8-hour count.

Based on a review of the crash data and traffic volume data collected in the field, FDOT District 4 staff concluded that an exclusive northbound left-turn lane was needed at the SR 76/SW Tropical Avenue intersection. The need for an exclusive southbound right-turn lane was also identified by FDOT District 4 staff as part of their study.

Deceleration Length (L_D)

As indicated in the *2010 FDOT Design Standards* (*31*), FDOT uses a standard deceleration length based on the design speed (shown in Figure 28). The total deceleration distance is 350 ft based on a design speed of 55 mph under rural conditions. This distance includes a standard 50-ft taper, as described in the subsequent section on taper length.

Taper Length (L_T)

FDOT uses a taper length of 50 ft in all cases. This relatively short taper length increases the number of queued vehicles that can be stored in the turn lane (relative to a longer taper) and provides drivers with a distinct visual cue with respect to the transition from the through lane(s) to the left-turn lane.

Queue Storage Length (L_S)

For the subject intersection, FDOT used 50 ft of storage (or the queue length equivalent of two passenger cars). Given the long deceleration distance at this location, sufficient queue storage is available if needed during times of peak traffic flow.

Design Result

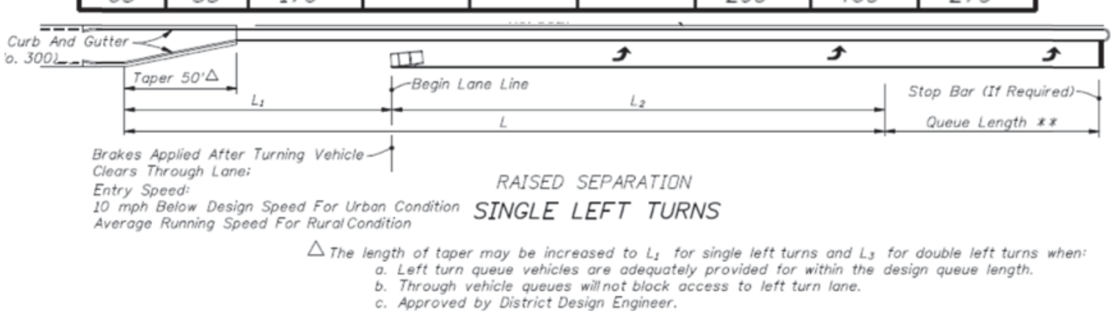
Figure 29 illustrates the final design configuration of the SR 76/SW Tropical Avenue intersection with implementation of the northbound left-turn lane on SR 76.

Design Example #11: Installation of Exclusive Left-Turn Lanes and a Traffic Signal at an Unsignalized Intersection Between a Local Street and a State Highway

(*Note: The intersection location in this design application has been made anonymous at the request of the contributing agency. Street names have been changed.*)

TURN LANES • CURBED AND UNCURBED MEDIANS

Design Speed (mph)	Entry Speed (mph)	Clearance Distance L_1	URBAN CONDITIONS Brake To Stop Distance L_2	URBAN CONDITIONS Total Decel. Distance L	URBAN CONDITIONS Clearance Distance L_3	RURAL CONDITIONS Brake To Stop Distance L_2	RURAL CONDITIONS Total Decel. Distance L	RURAL CONDITIONS Clearance Distance L_3
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'



(Source: *2010 FDOT Design Standards, Index 301*. Used by permission.)

Figure 28. Deceleration distance values from* 2010 FDOT Design Standards *(*31*).

(Source: Google Earth™ mapping service.)

Figure 29. SR 76/SW Tropical Avenue intersection: final condition.

(Source: Google Earth™ mapping service.)

Figure 30. Route 90/Spenlow Drive intersection: condition before installation.

Context

Conservatory Parkway is a two-way, east-west suburban arterial that serves as a border between two adjoining cities and is part of the state highway system as Route 90. The cross section of Route 90 is one 12-ft travel lane and a 10-ft shoulder in each direction. The posted speed limit on this section of Route 90 is 60 mph. Access points to Route 90 are minimal because there are few developed properties along this corridor. One of the few intersections on Route 90 is that of Spenlow Drive, a four-lane divided suburban collector that provides access to several properties in a mixed-use development, including a hospital, banks, and several medical and professional buildings. The mixed-use development is bordered on either side by multiple residential subdivisions abutting Spenlow Drive.

The intersection of Route 90 and Spenlow Drive was originally built as a T-intersection, with southbound Spenlow Drive terminating at Route 90. As development increased, an extension to Spenlow Drive was built on the south side of Route 90, resulting in a traditional 90-degree four-leg intersection (see Figure 30). The approaches on Spenlow Drive were stop controlled.

Design Request

Engineers from the two cities and the state DOT monitored this intersection for several years as the adjacent development increased and traffic volumes increased correspondingly. The speed limit on Route 90 was originally 65 mph, but as a response to the increasing volumes, it was reduced to its current 60 mph approximately 1 year prior to the start of construction of the turning lanes. The engineers had originally studied whether the intersection met one or more warrants for a traffic signal; after the fourth leg of the intersection was built, more studies were conducted, and results indicated that the 8-hour, 4-hour, and peak-hour volume warrants were indeed met. However, the state DOT does not typically signalize a high-speed two-lane highway that does not have a left-turn lane. So the DOT agreed to allow the intersection to be signalized if left-turn lanes were added, and both cities began working toward signalizing the intersection and adding a left-turn bay. Prior to any projects being let, however, the DOT issued a statewide call for safety projects, and local officials submitted a left-turn lane project at this intersection. The project was selected for the safety funding, so only the cost of the traffic signal remained for the two cities to split.

Design Considerations and Analysis

The primary design considerations evaluated by the DOT were the high speeds on Route 90 and access to the nearby hospital. Right-of-way was available, and a nominally straight, level alignment at the intersection facilitated a straightforward design. The high speeds on Route 90 required longer tapers and deceleration lengths than for roads with lower speeds. The hospital access is located approximately 500 ft west of the intersection, which necessitated special accommodation, so the length of the widened roadway was extended west of

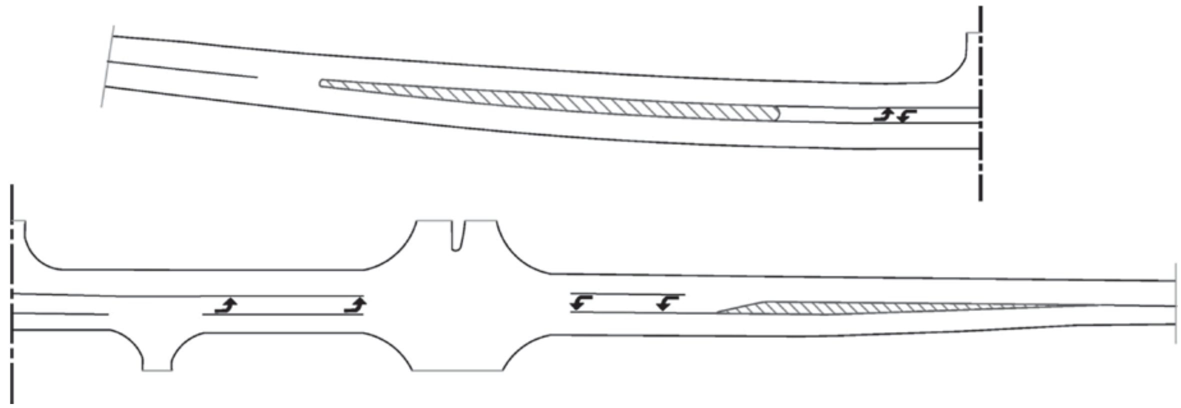


Figure 31. Pavement markings diagram for Route 90 at Spenlow Drive.

the hospital access to provide an eastbound turning lane for the hospital, upstream of the eastbound left-turn lane for the intersection. On the westbound approach to the intersection, the DOT used the typical lengths in their design manual. Specifically, the DOT addressed the following items as part of the design:

1. **Determine the deceleration length and taper length.** The DOT's roadway design manual provides a table of standardized lengths for turn lanes on high-speed roadways. For roadways with a design speed of 65 mph, the table calls for a taper length of 150 ft and a deceleration length of 715 ft, which were provided on the westbound approach to the intersection and used as the baseline values for the eastbound approach.
2. **Determine adjustments for hospital access.** Because the hospital access is within about 500 ft of the intersection, the full deceleration length could not be provided in the eastbound left-turn lane at the intersection, so the length was shortened to about 475 ft. Upstream of the hospital access, a standard widening taper was applied to add the center lane, and a turning lane of approximately 400 ft was installed for the hospital access. This turning lane is striped as a TWLTL to distinguish it from the left-turn lane provided for the intersection.

Design Result

The intersection is completely within the city on the south side of Route 90, but the majority of the traffic travels into/out of the city on the north. For this reason, the northern city was willing to pay for half of the improvement costs. Additionally, as mentioned previously, the DOT successfully secured safety fund money to cover the cost of adding the left-turn lanes, so the resulting project was a group effort among the three agencies. The cities paid for the design, and all three agencies reviewed the plans. The DOT let the project with an Advanced Funding Agreement, with the northern city to pay 100 percent of the signal cost. The southern city had a separate contract with the northern city to reimburse it for 50 percent of the signal cost. Figure 31 illustrates the pavement markings in the final design configuration with implementation of the exclusive left-turn lanes on Route 90, with the additional provision for turns into the hospital to the west of the intersection.

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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