Florida Intersection Design Guide 2013

For New Construction and Major Reconstruction of At-Grade Intersections on the State Highway System

Florida Department of Transportation
Office of Roadway Design

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Acknowledgements

The Florida Intersection Design Guide was originally published in 2002 and is maintained by the Florida Department of Transportation (FDOT) to identify requirements for intersection design projects and to provide guidelines for choice where alternatives exist. The development was overseen by the FDOT State Roadway Design Office. The technical coordinator was James A. Mills, P.E., Roadway Design Engineer. Robert F. Quigley, P.E. and Larry Bodiford also of the State Roadway Design Office provided noteworthy technical assistance with reviews and the preparation of certain figures used in the final document.

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1 INTRODUCTION

The design of at-grade intersections requires strict conformance with standard practice, combined with the experience and creativity of the designer in selecting and applying the most appropriate treatment to accommodate each traffic movement. Uniformity is an important ingredient of intersection design because it is essential that all road users encounter familiar conditions at each intersection. Uniform standards and principles also serve to promote intersection treatments that have proven successful and have been accepted by transportation professionals and road users.

On the other hand, each intersection may have unique features that distinguish it in some way from other intersections. In addition, there are legitimate differences in local preferences that have created a set of equally acceptable alternatives for some treatments. This creates a tradeoff between uniformity and flexibility. Clearly, the most appropriate design policy is one that sets forth the standards and principles that must be observed and provides some latitude for choice in areas where choice can be offered.

The purpose of this document is to identify the mandatory requirements and to provide guidelines for choice where alternatives exist. The mandatory requirements are collected from several sources that are recognized by the Florida Department of Transportation (FDOT). The guidelines represent a combination of material from authoritative references and research reports combined with the consensus of a broad based team of transportation professionals.

1.1 INTERSECTION DESIGN REQUIREMENTS AND OBJECTIVES

The guidelines presented in this document are based on the premise that the design of an intersection must conform in all respects to the provisions of the ***Florida Statutes*** and rules, plus all authoritative references that have been adopted as standards by the FDOT.

In addition, the design should be such that it provides:

1. Safe and convenient operation for all road users, including cyclists and pedestrians;
2. Proper accessibility for pedestrians with special needs;
3. Adequate capacity for peak-hour demand on all movements;
4. Adequate maneuvering space for design vehicles;
5. Resolution of conflicts between competing movements;
6. Reasonable delineation of vehicle paths;
7. Adequate visibility of conflicting traffic;
8. Storage for normal queuing of vehicles;
9. Appropriate access management application;

10. Minimum delay and disutility to all road users;
11. Proper drainage of storm water;
12. Accommodation for all utilities, both above and below the ground;
13. Necessary regulatory, warning and informational messages for all road users;
14. Suitable advance warning of all hazards;
15. Uniformity of treatment with similar locations; and,
16. Minimal consumption of resources.

1.2 THE DESIGN PROCESS

The product of the design process will be a set of design details presented on a plan. The principal governing document for plans preparation on the Florida State Highway System (SHS) is the FDOT ***Plans Preparation Manual (PPM)***. The overall process as described in the ***PPM*** is summarized in ***Figure 1-1***. Three steps are illustrated in this figure, including selection of design standards, development of design details and preparation of the plans.

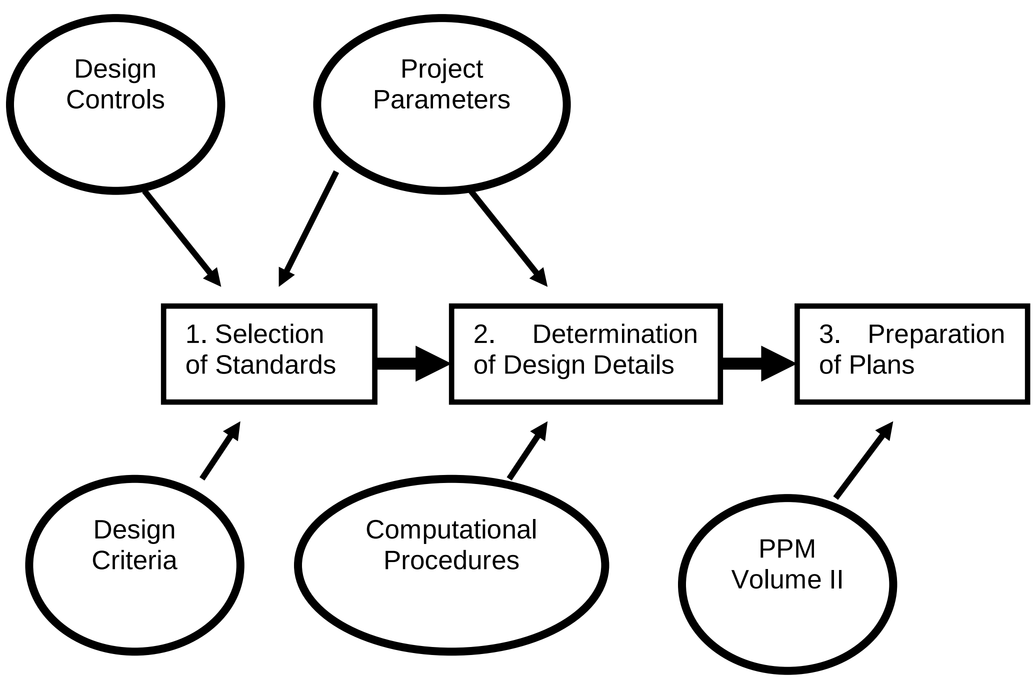

Figure 1-1 Design Process Steps and Information Flow

The information and procedures that support these steps is also illustrated in ***Figure 1-1***. The process is described as follows:

1.2.1 Selection of Standards

The specific values selected from the roadway design criteria become the design standards for a design project. There are three sources of supporting information shown in ***Figure 1-1***, including:

> Design criteria and references will be presented in boxes like this in the chapters that deal with individual design elements.

1. *Design Criteria*: Design criteria take the form of specific requirements, values or ranges of values or other conditions that are set forth in an authoritative reference such as the ***PPM***, the ***Manual on Uniform Traffic Control Devices (MUTCD), A Policy on Geometric Design of Highways and Streets (AASHTO Green Book)*** or the ***Florida Statutes***. Specific design criteria and their sources will be presented in more detail in the discussion of each of the intersection design elements.

2. *Design Controls*: Design controls are site-specific characteristics and conditions that influence or regulate the selection of the criteria for project standards. The identification and application of these controls are important parts of the design process. Examples of design controls include design year volumes, traffic composition, directional distribution, design speed, road user characteristics, etc. In general, the design controls are "givens" and as such, are not subject to selection by the designer.

> Example:
> The minimum pedestrian clearance time is based on an assumed walking speed of 3.5 ft/sec
> ***{MUTCD}***

3. *Project Parameters*: Project parameters are the properties or specific conditions of a project that require the modification of design standards. The designer is responsible for establishing and documenting any project parameters and the justification for deviations from project standards.

As an example of the above terminology and process, the ***MUTCD*** establishes the *criterion* of 4 ft/sec walking speed for determining the minimum pedestrian clearance time at a signalized crosswalk. The site-specific street width of 60 feet is a *control* that would lead to the *standard* of 15 seconds minimum pedestrian clearance time for the approach in question. The existence of a nearby residence for senior citizens could be recognized as a *project parameter* that would require a reduced pedestrian walking speed of, say, 3 ft/sec.

Note the presentation of the criterion in the box. This format will be used whenever practical to present specific criteria in later chapters. To avoid lengthy and verbose presentations associated with verbatim quotations, the criteria may be paraphrased and/or converted from metric format, as it was in the example above.

1.2.2 Development of Design Details

The design details may be determined directly from the standards in some cases. It may, however, be necessary to apply additional computational procedures using other design controls. To continue the example presented above, there is no criterion for the maximum length of a pedestrian clearance interval. The judgment of the designer could also be an input here and standards developed from other criteria, such as the maximum allowable cycle length, would also apply.

Project parameters may also influence the design details. For example, in the situation presented above, the existence of an adjacent railroad grade crossing would be a project parameter that could require that the pedestrian clearance display time be reduced below the standard when the signal operation is preempted by the arrival of a train. This, in turn could introduce a need for other design details such as special warning signs.

1.2.3 Preparation of Plans

When all of the design details have been determined they will be documented in the contract plans. This is a formal process governed by Volume II of the ***PPM,*** which deals exclusively with “Plans Preparation and Assembly”.

1.3 AUTHORITATIVE REFERENCES

The FDOT recognizes the following publications as authoritative references that provide criteria, guidelines and computational procedures for intersection design purposes:

1.3.1 References Governing Design Criteria

The FDOT ***Plans Preparation Manual (PPM)*** was introduced earlier as the principal governing document for plans preparation on the SHS. The ***PPM*** is divided into two volumes. Volume I describes the design criteria and process and Volume II deals with plans preparation and assembly details.

The ***Manual on Uniform Traffic Control Devices (MUTCD)***. The ***MUTCD*** contains standards for traffic control devices that regulate, warn and guide road users along the highways and byways in all 50 States. The current version has been approved by the Federal Highway Administrator as the National Standard in accordance with Title 23 U.S. Code. The effective date for conformance is January 17, 2004.

The information in the ***MUTCD*** is presented in four categories:

1. **Standard** - a statement of required, mandatory or specifically prohibitive practice regarding a traffic control device. The verb “shall” is typically used. Standards are sometimes modified by Options.
2. **Guidance** - a statement of recommended, but not mandatory, practice in typical situations, with deviations allowed if engineering judgment or engineering study indicates the deviation to be appropriate. The verb “should” is typically used. Guidance statements are sometimes modified by Options.
3. **Option** - a statement of practice that is a permissive condition and carries no requirement or recommendation. Options may contain allowable modifications to a Standard or Guidance. The verb “may” is typically used.
4. **Support** - an informational statement that does not convey any degree of mandate, recommendation, authorization, prohibition or enforceable condition. The verbs “shall”, “should” and “may” are not used in Support statements.

The ***MUTCD*** is divided into several parts, each of which covers a particular aspect of traffic control. In addition to the general provisions of Part 1, the following parts have the strongest application to the design of traffic controls for intersections:

Part 2: Signs,

Part 3: Markings,

Part 4: Signals,

Part 7: School areas and

Part 9: Bicycle Facilities.

The FDOT has adopted the ***MUTCD*** as a standard that applies to all roads in the State.

FDOT ***Design Standards****:* This document contains a series of index sheets with drawings that prescribe detailed requirements for construction and maintenance of highway facilities in Florida. The index sheets that apply to intersection design will be identified in each chapter and discussed as necessary.

FDOT ***Standard Specifications for Road and Bridge Construction*** (also known as ***FDOT Specifications***): This document is written for the bidder on state construction projects. It is oriented primarily to construction methods and materials. The sections that cover traffic control equipment and materials, roadway lighting, landscaping, etc. all have application to intersection design.

A Policy on Geometric Design for Streets and Highways: Published by the American Association of State Highway and Transportation Officials (AASHTO), this document is also known as the ***AASHTO Green Book***. It contains a comprehensive compilation of criteria used in geometric design of all facilities.

The ***Florida Manual of Uniform Minimum Standards for Design, Construction and Maintenance for Streets & Highways****,* also known as the "***Florida Greenbook***", sets forth minimum criteria for new construction projects off the State Highway System.

The FDOT ***Traffic Engineering Manual (TEM)***: The purpose of this manual is to provide traffic engineering standards to be used on the State Highway System by the Department's District Traffic Operations Offices.

Florida Administrative Code Chapter 14-97, State Highway System Access Management Classification System and Standards: This rule chapter sets forth an access control classification system and access management standards to implement the State Highway System Access Management Act of 1988.

Americans with Disabilities Act (ADA): This law provides for criteria and standards required of all construction projects in order to accommodate human disabilities.

1.3.2 References Containing Design Guidelines

NCHRP Report 672, Roundabouts: An Informational Guide (2010): This guide has been officially adopted as the primary resource for the design of Roundabouts in the State of Florida.

Trail Intersection Design Handbook (1996)*:* Describes the unique characteristic of trail-highway intersections, summarizes the literature on bicycle and pedestrian trails, identifies design principles and elements and provides guidelines based on research and case studies.

***The AASHTO Guide for the Development of Bicycle Facilities (1999)**:* provides guidance on intersection treatments for a variety of bicycle facilities and has gained widespread national acceptance.

***The AASHTO Guide for the Planning, Design, and Operation of Pedestrian Facilities (2004)**:* provides guidance on the planning, design and operation of pedestrian facilities along streets and highways.

***The Median Handbook**:* provides a complete technical guide to median decisions. This reference contains guidance on design, queuing, U-turn placement and safety benefits associated with medians.

***NCHRP Report 457**:* Evaluating Intersection Improvements: an Engineering Study Guide. This guide describes the engineering study process for evaluating the operational effectiveness of various engineering improvements. It also shows how capacity analysis and traffic simulation models can be used to assess the operational impacts of those improvements.

1.3.3 References Prescribing Design and Analysis Procedures

The ***Highway Capacity Manual (HCM):*** The provision of adequate capacity is a primary design objective. It is essential; therefore, that an acceptable procedure be available for estimating the capacity of any given design. The ***HCM*** prescribes capacity estimation procedures for all at-grade facilities. The ***HCM*** procedures are based on a combination of traffic modeling, field data and expert consensus. As such, they are widely accepted by transportation professionals and are recognized by the FDOT as the preferred capacity analysis technique. One feature that has contributed to the popularity of the ***HCM*** is the availability of publicly supported software that provides a faithful implementation of the procedures.

The ***Highway Safety Manual (HSM)***: Published by the American Association of State Highway and Transportation Officials (AASHTO), this manual provides safety knowledge and tools in a useful form to facilitate improved decision making based on safety performance.

The FDOT ***Manual on Uniform Traffic Studies (MUTS)***: This document prescribes standard collection and analysis techniques for the field data required to support design decisions. Specific studies and procedures will be described in the chapters to which they apply.

1.4 LOCAL POLICIES, PREFERENCES AND PRACTICES

Local agencies have assumed a stronger influence in design decisions affecting intersections in recent years. The design goals of the community should be considered in all intersection design activities. Before any design is finalized, the urban design criteria contained in local government comprehensive plans, land development regulations and Metropolitan Planning Organization (MPO) policies related to transportation should be investigated. These criteria differ widely from region to region. They cover a variety of topics, with particular emphasis on multi-modal aspects of safety, accessibility, air quality and aesthetics.

1.5 INTERSECTION DESIGN ISSUES

The term "issue" is used in many different contexts. Webster's dictionary offers nine separate definitions of this word. The applicable definition is "a matter that is in dispute between two or more parties". For purposes of this guide, an issue will be considered as any question that has multiple legitimate answers arising from and supported by different perspectives.

From the example already presented, "What is the minimum pedestrian clearance time at a signalized intersection?" is a question and not an issue, because accepted criteria and computational techniques exist for determining this requirement. On the other hand, "What is the minimum pedestrian WALK time?" is an issue because the ***MUTCD*** suggests a value between four and seven seconds and there is some disagreement on the interpretation of this requirement.

One of the objectives of this guide is to resolve such issues; in other words, to identify and recommend the most appropriate answer to all issue-related questions. In some cases, a set of equally acceptable alternatives will be presented. Guidelines are offered to facilitate choices between alternatives when they exist. The recommendations may be based on the experience of other agencies, modeling and analysis or consensus of our team of transportation professionals. Issues that cannot be resolved will be reported as such. All issues will be addressed in the chapters to which they apply.

Chapter 2

Intersection Design Concepts

Tables

Figures

2 DESIGN CONCEPTS

It was pointed out in Chapter 1 that intersection design is founded on an extensive set of design criteria, controls, project parameters and standards. The design process also makes use of several concepts that apply to intersections in general. This chapter identifies the broad considerations that must be understood by designers. Subsequent chapters will provide additional details.

Intersections are defined by both their physical and functional areas as illustrated in ***Figure 2-1*** and ***Figure 2-2***. The functional area of an intersection extends both upstream and downstream from the physical area and includes any auxiliary lanes and their associated channelization:

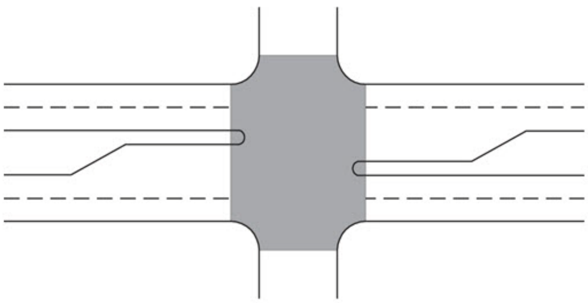

Figure 2-1 Intersection Physical Definition

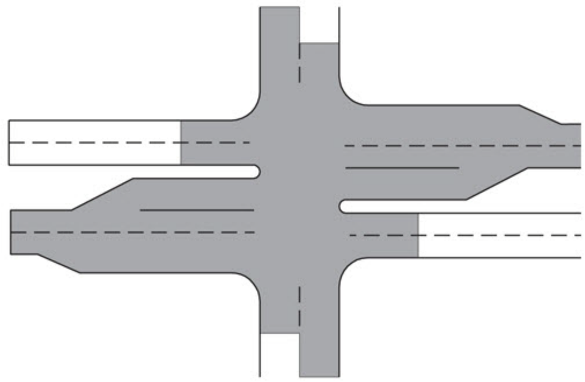

Figure 2-2 Intersection Functional Design

The functional area on the approach to an intersection or driveway consists of three basic elements: (1) perception-reaction-decision distance, (2) maneuver distance and (3) queue-storage distance. These elements are shown in ***Figure 2-3***. The maneuver distance includes the length needed for both braking and lane changing when there is a left or right turning lane. In the absence of turn lanes, the maneuver distance is the distance to brake to a comfortable stop. The storage length should include the most distant extent of any intersection-related queue expected to occur during the design period.

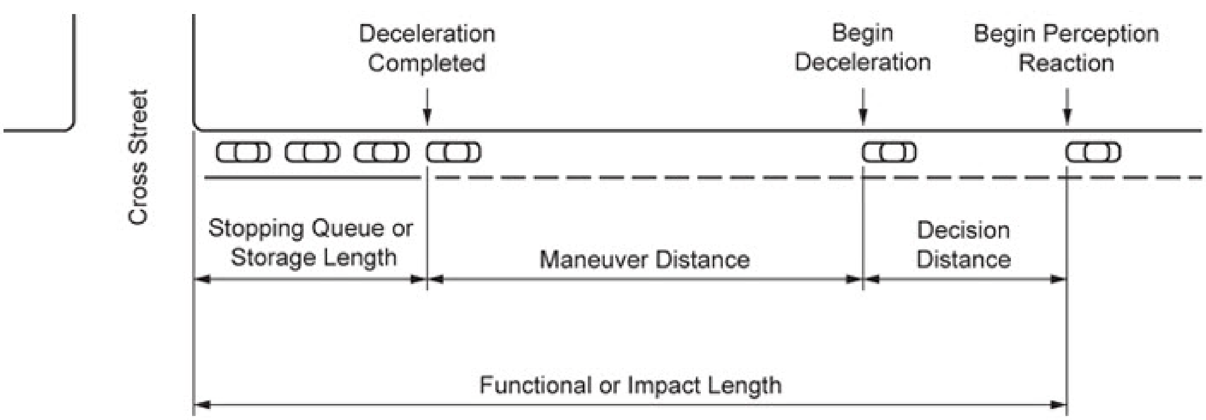

Figure 2-3 Elements of the Functional Area of an Intersection

2.1 INTERSECTION CHARACTERISTICS

The characteristics of an intersection include a set of factors that are involved in some way in the intersection design process. Some characteristics act as controls, while others are the result of design decisions. A summary of these characteristics is presented in ***Table 2-1***.

Physical characteristics	Traveled roadway Curbs Sidewalks Medians Islands Inscribed circle diameter Drainage features Physical obstacles
Operational characteristics	Lane configuration and usage Traffic control mode Pedestrian control provisions Lane delineation Turn prohibitions Crosswalk configuration Signal phasing and timing Accessibility features
Traffic characteristics	Vehicular volumes Composition Peaking characteristics Pedestrian volumes Bicycle volumes Safety Performance
Site characteristics	Roadway classification Site location Roadside development Institutional proximity (schools, etc.)
Road User Characteristics	Age Special Requirements

Table 2-1 Summary of Intersection Characteristics

2.2 RESOLUTION OF CONFLICTS BETWEEN COMPETING MOVEMENTS

In the interest of safety, the conflict between two competing traffic movements must be resolved by a traffic control discipline that gives one movement priority over the other. When some movements are heavy, the priority must be assigned, alternated or distributed in some manner or at least one of the movements will fail to provide adequate service.

2.2.1 The Nature of Conflicts

Most conflicts at intersections occur when two vehicles compete with each other for right of way. It is, however, important to recognize the conflicts that occur between two different types of road users. Inter-user conflicts may be characterized as:

1. Vehicle-pedestrian conflicts: The ***Florida Statutes*** assign the right of way to pedestrians crossing in crosswalks, subject to traffic control signals, where existing. Pedestrians crossing outside of crosswalks must yield to vehicles. Most pedestrian activity at intersections will be in crosswalks.
2. Vehicle-bicycle conflicts: The ***Florida Statutes*** considers a bicycle to be a vehicle. In the traveled roadway, a bicycle is assigned all the rights and responsibilities applicable to the driver of any other vehicle. When crossing a roadway at crosswalks and riding upon and along sidewalks, the ***Florida Statutes*** assigns the cyclist all the rights and duties of a pedestrian. The accommodations for bicycles at intersections must recognize these assignations.
3. Bicycle-pedestrian conflicts: Because they are assigned all the rights and responsibilities applicable to the driver of any other vehicle, bicycles on roadways are subject to the same rules indicated above for vehicle-pedestrian conflicts. When bicycles are on pedestrian facilities such as sidewalks, crosswalks, etc. they are required to yield to pedestrians.

2.2.2 Traffic Control Disciplines

The method of resolving conflicts between any two movements is referred to as the traffic control discipline. The following traffic control disciplines are commonly employed at intersections:

1. Right of Way Rule: The right of way rule applies in the absence of any other traffic control device. ***Florida Statutes, Section 316.121*** assigns the right of way at an uncontrolled intersection to a vehicle that has already entered the intersection, the first arriving vehicle, or to the vehicle on the right in cases of simultaneous arrival.
2. Fixed Priority: There are two cases of fixed priority covered in the ***Florida Statutes***. ***Florida Statutes, Section 316.122*** assigns right of way to through and lawfully passing vehicles in conflict with left turns and ***Florida Statutes, Section 316.123*** governs vehicles on approaches controlled by stop or yield signs.

3. Roundabouts: Vehicles entering a roundabout must yield to vehicles already in the circulatory roadway. The ***MUTCD*** specifies that *yield signs shall be posted* at the entrance to a roundabout to assign right-of-way. ***Florida Statutes, Section 316.088*** specifies that a vehicle passing around a rotary traffic island shall be driven only to the right of such island.

4. Alternating Priority (signals): The assignment of right of way by traffic signals is covered in ***Florida Statutes, Section 316.075***.

5. Weaving Movements: Vehicles proceeding in the same direction may also conflict with each other if their respective origins and destinations cause their paths to cross. ***Florida Statutes, Section 316.085*** places the burden on drivers changing lanes to ensure that the movement may be made in safety. Weaving movements are used by design for resolving conflicts at roundabouts and some types of freeway interchanges.

6. Grade Separation: The conflict between movements is eliminated if the movements take place at different levels. Grade separations are typically used to resolve conflicts between major movements at interchanges. It is important to note, however, that, except in the case of extremely complex interchanges, one or more of the other disciplines mentioned above will also be required to resolve some of the conflicts for turning movements.

2.2.3 Traffic Control Modes

A combination of traffic control disciplines will be found at most intersections. It would, however, be very confusing to describe the traffic control at an intersection by enumerating the disciplines that are applied to each set of movements. Instead, the intersection control is characterized by a "mode" that reflects the predominant traffic control discipline for the major movements.

One of the first steps in the intersection design process is to choose the most appropriate control mode from the following alternatives:

1. Uncontrolled: This is the default mode and the only mode that requires no action to establish. In this case, the right-of-way rule applies. Because of safety concerns, there are very few uncontrolled intersections in Florida, especially on state roadways.

2. Yield or Stop Signs: The right-of-way rule can be modified by placing YIELD signs or STOP signs on one or more approaches. Such placement requires minimal justification and there are no numerical warrants to be applied. The ***MUTCD*** suggests that YIELD or STOP signs should be used when one route is clearly more important than the other and when conditions indicate the need.

STOP signs shall not be used in conjunction with any traffic control signal except:

1. If the signal indication for an approach is flashing red at all times.
2. If a minor street or driveway does not require separate traffic signal control because an extremely low potential for conflict exists.
3. If a channelized turn lane is separated from the adjacent traffic lanes by an island and the channelized turn lane is not controlled by a traffic control signal.

{MUTCD}

3. All-way stop control: The application of this mode is subject to an engineering study considering numerical traffic volume guidance presented in the ***MUTCD***. Additional considerations include locations with high crash rates and locations where traffic signals are warranted but have not yet been installed.

4. Signal Control: The ***MUTCD*** specifies that an engineering study of traffic conditions, pedestrian characteristics and physical characteristics of a location shall be performed to determine whether installation of a traffic control signal is justified. The study shall include an analysis of factors related to the existing operation and safety at the subject location and the potential to improve these conditions. The applicable factors are contained in nine traffic signal warrants that may be used to justify the installation of a traffic signal. The ***MUTCD*** suggests that a traffic control signal should not be installed unless one or more of the warrants are met and that an engineering study indicates that installing a signal will improve the overall safety and/or operation of the intersection. Before a decision is made to install a traffic control signal, consideration shall be given to implementation of other remedial measures. The ***MUTCD*** discourages the installation of a signal if it would seriously disrupt progressive traffic flow.

> **MUTCD Warrants for Signal Installation**
>
> 1. Eight-Hour Vehicular Volume
> 2. Four-Hour Vehicular Volume
> 3. Peak Hour
> 4. Pedestrian Volume
> 5. School Crossing
> 6. Coordinated Signal System
> 7. Crash Experience
> 8. Roadway Network
> 9. Intersection Near a Grade Crossing
>
> "The satisfaction of a traffic signal warrant or warrants shall not in itself require the installation of a traffic control signal."

5. Roundabout: Due to substantial safety characteristics, and potentially significant operational and capacity advantages, the modern Roundabout is a preferred traffic control mode for any new road or reconstruction project. Roundabouts should be considered as an alternative to all the other traffic control modes described above. The FHWA's adopted NCHRP Report 672 – ***Roundabouts: An Informational Guide***, has been adopted by the State of Florida for the implementation of Roundabouts, subject to specific conditions provided in Chapter 7 of this Guide, Plans Preparation Manual, Florida Greenbook, and FDOT Design Standards.

2.2.4 Intersection Control Mode Selection Procedures

The FDOT ***MUTS*** prescribes justification studies for traffic signals. The ***MUTCD*** suggests (as guidance) that vehicular delay and the frequency of some types of crashes are sometimes greater under traffic signal control than under STOP sign control and encourages consideration of alternatives to signalization.

The ***MUTCD*** suggests the following alternatives to signalization:

1. Installing signs along the major street to warn road users approaching the intersection;
2. Relocating the stop line(s) and making other changes to improve the sight distance at the intersection;

3. Installing measures designed to reduce speeds on the approaches;
4. Installing a flashing beacon at the intersection to supplement STOP sign control;
5. Installing flashing beacons on warning signs in advance of a STOP sign controlled intersection on major and/or minor-street approaches;
6. Adding one or more lanes on a minor-street approach to reduce the number of vehicles per lane on the approach;
7. Revising the geometrics at the intersection to channelize vehicular movements and reduce the time required for a vehicle to complete a movement, which could also assist pedestrians;
8. Revising the geometrics at the intersection to add pedestrian median refuge islands and/or curb extensions;
9. Installing roadway lighting if a disproportionate number of crashes occur at night;
10. Restricting one or more turning movements, perhaps on a time-of-day basis, if alternate routes are available;
11. If the warrant is satisfied, installing multi-way STOP sign control;
12. Installing a pedestrian hybrid beacon (see Chapter 4F) or In-Roadway Warning Lights (see Chapter 4) if pedestrian safety is the major concern;
13. Installing a roundabout; and
14. Employing other alternatives, depending on conditions at the intersection.

2.3 ESTIMATION OF CAPACITY

The provision of adequate capacity is a primary design objective. It is essential therefore that an acceptable procedure be available for estimating the capacity of any given design. The ***Highway Capacity Manual (HCM)*** prescribes capacity estimation procedures for all interrupted flow facilities. The ***HCM*** procedures are based on a combination of traffic modeling, field data and expert consensus. As such, they are widely accepted by transportation professionals and are recognized by FDOT as the preferred capacity analysis technique. Software is available that provides a faithful implementation of the ***HCM*** procedures.

The traffic control modes for which capacity estimation procedures are defined in the HCM include:

- Signalized Intersections
- Two-Way Stop Controlled Intersections
- All-Way Stop Controlled Intersections
- Roundabouts

2.3.1 Capacity and LOS at Intersections

The procedures for analyzing capacity and LOS differ among control modes. Each of the control modes will therefore be discussed separately.

2.3.1.1 Two-Way STOP-Controlled Intersections

The level of service at unsignalized intersections is also defined in terms of control delay, but the thresholds and computational procedures are different than those applied at signal. The movement of traffic at a stop or yield sign is modeled in Chapter 19 of the ***HCM*** as a gap acceptance process. The operation of a two-way stop controlled intersection is modeled through the use of the following parameters:

a. The *critical headway*, which is defined as the minimum time interval in the major street traffic stream that allows intersection entry for one minor-street vehicle, and

b. The follow-up headway, which is defined as the time between the departure of one vehicle from the minor street and the departure of the next vehicle using the same major-street headway, under a condition of continuous queuing on the minor street.

For the automobile mode analysis, the methodology addresses a number of special circumstances that may exist at two-way stop controlled intersections, including:

- Two-stage gap acceptance,
- Approaches with shared lanes,
- The presence of upstream traffic signals, and
- Flared approaches for minor-street right-turning vehicles

The procedure also provides for ranking of each of the movements at an intersection, and computation of critical movements' control delay. Because the delay for major-street vehicles at a two-way stop controlled intersection is zero, delay is only calculated for the minor street movements, and is used only in comparison with other traffic control options.

2.3.1.2 All-Way STOP-Controlled Intersections

Field observations indicate that standard four-leg all-way stop controlled intersections operate in either a two-phase or four-phase pattern based on intersection geometry, primarily the number of lanes on each approach. The operation of an all-way stop controlled intersection is modeled through the use of the following parameters:

a. The *saturation headway*, which is defined as the time between departures of successive vehicles on a given approach, assuming a continuous queue,

b. The *departure headway*, which is defined as the average time between departures of successive vehicles on a given approach, and

c. The *service time*, which is the average time spent by a vehicle in the first position waiting to depart. It is equal to the departure headway minus the time it takes a vehicle to move from second position into first position (the move-up time).

2.3.1.3 Roundabouts

Research has demonstrated unequivocally that under appropriate conditions, Roundabouts are the highest performing traffic control mode in terms of capacity (intersection delay). Chapter 21 of the ***HCM*** presents concepts and procedures for calculating delay at Roundabouts for direct comparison with other traffic control modes. The methodology is greatly simplified in that there are limited parameters pertaining to movements: Entry flow rate, Conflicting flow rate, and Exit flow rate.

Analysis procedures exist for various roundabout configurations including single-lane, multi-lane and spiral or “Turbo” Roundabouts, as well as, yielding and non-yielding right-turn bypass lanes. Although procedures account for pedestrian impacts to vehicular delay, currently there is no model specifically designed for vehicular impacts to pedestrian quality of service at roundabouts.

2.3.1.4 Signalized Intersections

The basic function of a traffic signal is to resolve conflicts between vehicles, cyclists and pedestrians that are competing for time and space at the intersection. The design objective is to ensure that the manner in which the conflicts are resolved provides the desired level of safety, capacity and performance.

The LOS at a signal is defined in the ***HCM,*** Chapter 18, Signalized Intersections in terms of the average control delay experienced by each vehicle on the approach. Threshold values are provided to distinguish among levels of service.

The following terminology is used to quantify the signal operation:

1. Traffic volume, v, (vph);
2. Saturation flow rate, s (vphg);
3. Flow ratio, y, calculated as v/s;
4. Effective green time, g (sec);
5. Cycle length, C (sec);
6. Green ratio, calculated as g/C;
7. Capacity, c, calculated as sg/C;
8. Degree of saturation, X, calculated as v/c, i.e., vC/sg and
9. Control delay, D, estimated by the procedure prescribed by the ***HCM***.

Further complexity is introduced by such features as permitted left turns that must yield to oncoming traffic, shared lanes for through and left turning movements, multi-phase operation, etc. The ***HCM*** prescribes procedures to deal with all of these complications.

The capacity, c, of an approach to a signalized intersection is determined by multiplying the saturation flow rate by the proportion of time that the signal controlling the approach is effectively green. *Effective green* is defined as the total phase time for the approach (green + yellow + all-red) minus the *lost time* associated with starting and stopping the movement.

$c = sg/C$, where

- s is the saturation flow rate (vphg);
- g is the effective green time per cycle (sec) and
- C is the cycle length (sec).

2.4 INTERSECTION DELAY

Delay is an important measure of performance at an intersection. Delay contributes to the motorist's operating cost and perception of the quality of service. It may be expressed in two ways:

1. The *unit delay* (sec/veh), which is related to the motorist's perception of disutility at an intersection.
2. The *total accumulated delay* (veh-hours), which is related more to the economic performance of an intersection. One vehicle-hour of delay is accumulated when one vehicle is delayed for a full hour, 3600 vehicles are delayed for one second each, etc.

The total accumulated delay per hour may be determined as the product of the hourly volume times the average unit delay per vehicle.

The unit delay may be divided into four components. Each component is associated with a characteristic that, if eliminated, would eliminate the delay. The four components are:

1. Incident delay: The additional delay caused by the occurrence of an incident that reduces the capacity of the facility.
2. Control delay: The delay imposed by the traffic control. This component may be further characterized by the mode of operation, i.e., signal delay, stop sign delay or roundabout delay.
3. Traffic delay: The delay caused by interaction between vehicles in an uninterrupted traffic stream. This is generally mid-link delay caused by a reduction in speed.
4. Geometric delay: The delay caused by the geometric design feature, generally resulting from reduced speed of vehicles making turning movements.

To avoid overlapping definitions, the delay components are represented above in a hierarchical order. For example, an incident might increase the delay at a traffic signal and increased traffic volumes might increase the delay due to an incident. The hierarchy dictates that all delay that would be eliminated if the incident were eliminated is considered as incident delay. Thus, the unit delay per vehicle may be represented as the sum of these four components.

2.5 COMPONENTS OF INTERSECTION OPERATION

The operation of an intersection may be represented by four interacting components: the road user, the vehicle, the roadway and the traffic control devices, as illustrated in ***Figure 2-4***. Each of these components imposes its own set of requirements and constraints on the designer. Each component will be considered separately in terms of its requirements and constraints.

2.5.1 Requirements and Constraints of Road Users

Road user characteristics establish time-critical controls, such as advance placement distances for warning signs. They also influence the need for special accessibility features at intersections. The following considerations apply to road user requirements and constraints:

Perception and Reaction Time: The perception of and reaction to a continuous series of visual and audio cues that are part of the driving task involve four actions on the part of the driver, characterized as: perception, identification, emotion and volition. The time taken by the sequence of these events is commonly referred to as the PIEV time or perception-reaction time and it is defined as the total time taken for the driver to react to a stimulus.

PIEV time is a function of many factors. For intersection design purposes, a value of 1.0 second is commonly used to account for the driver's reaction to a signal change interval and 2.5 seconds is generally applied to more passive stimuli, such as a fixed warning sign.

Visual Acuity and Driving: Drivers are normally tested only for static visual acuity, that is, the ability to see stationary objects and legend messages. However, this is not the one that most dramatically affects the driving tasks. Other important measures are dynamic visual acuity, depth perception, glare recovery and peripheral vision. The three primary fields of vision that affect the driving task are: the field of clear or acute vision, the field of fairly clear vision and the field of peripheral vision.

Human Error: Improper operation and accidents may occur as a result of information-handling errors. These errors can be due to road user deficiencies and situational demands.

Driving Task: Driving comprises many sub-tasks, some of which must be performed simultaneously. The three major sub-tasks are:

- Control – Keeping the vehicle at a desired speed and heading within the lane. This requires the driver to be aware of numerous parameters including the vertical and horizontal alignment, width of the lane, vehicle size and performance characteristics, speed limits, etc. This subtask is normally performed almost automatically, and is heavily influenced by driver expectancy. Consistency in design parameters such as lane width, pavement markings, presence of shoulders, etc., affect the complexity of driver control.
- Guidance – Interacting with other vehicles (following, passing, merging, etc.) by maintaining safe following distance and by following markings, traffic control signs and signals. Highway design and traffic operations have the greatest effect on guidance. Driver performance can be improved by paying proper attention to lane placement, traffic conditions,, traffic control signs, signals, overtaking, passing and other guidance activities, (i.e., merging, lane changing, avoidance of pedestrians, etc.)
- Navigation – Following a path from origin to destination by reading guide signs and using landmarks. This is the most complex of the three sub-tasks of driving, and defines the overall objective of driving. Guide signs play a major role in assisting with navigation.

Cyclist Characteristics: Knowledge of bicycle dimensions, operating characteristics and requirements is also necessary for providing adequate bicycle facilities. These factors determine acceptable turning radii, grades and sight distances. Some of the measures that need to be considered to enhance the bicycle traffic safety and capacity at an intersection include:

1. Provision of bicycle lanes, either designated or undesignated;
2. Paved shoulders;
3. Wide outside traffic lane if no shoulder is present;
4. Bicycle-safe drainage grates;
5. Manhole covers adjusted to the grade and
6. Maintaining a smooth, clean riding surface.

Pedestrian Characteristics: The safety of pedestrians, especially at intersections, is a very important consideration in the highway design process. Important pedestrian characteristics are: pedestrian crossing volumes, walking speeds and gap acceptance characteristics at crossing locations.

Special Needs of Road Users: Road users with physical, visual or hearing disabilities introduce controls that could modify standards (e.g., clearance time requirement). They also introduce the need for accessibility features in pedestrian detection, displays, etc. These features are covered in more detail in Chapter 4.

2.5.2 Requirements and Constraints of Motor Vehicles

The designer must be aware of the roadway space requirements and the performance limitations of a typical vehicle. The following considerations apply to vehicle requirements and constraints:

1. The Design Vehicle: AASHTO has established a set of ten “design vehicles” with standard physical dimensions. These are used to determine a variety of geometric highway features, such as lane widths, minimum curb and corner radii and minimum turning radius.
2. Acceleration Performance: The difference in acceleration capability between a car and a truck is substantial and this is a major cause of inefficiency in mixed traffic streams. Important factors to consider are: distance traveled during acceleration, upgrades and crawl speeds.
3. Braking Performance: A vehicle’s braking performance is one of the most critical factors in highway safety and design and it is related to the vehicle’s braking system, type and condition of the tires and the condition and type of roadway surface. Almost every aspect of traffic-system design and operation is determined by the time and distance required to stop a vehicle or so called braking distance. Elements that determine braking distance are: the initial and final speed of vehicle, coefficient of forward rolling or skidding friction and grade. ***Florida Statutes, Section 316.262*** prescribes the performance ability of motor vehicle brakes.

2.5.3 Roadway Requirements and Constraints

The roadway has its own set of requirements and constraints that must be considered. The most significant points include:

Roadway Utilization: *The following sections of the **Florida Statutes** govern roadway utilization:*

316.0765 Lane direction control signals
316.081 ***Driving on right side of roadway;***
316.087 Further limitations on driving to left of center of roadway;
316.088 One-way roadways and rotary traffic islands;
316.089 Driving on roadways laned for traffic;
316.090 Driving on divided highways;
316.130 Pedestrians; traffic regulations;
316.151 Required position and method of turning at intersections;
316.1515 Limitations on turning around. (i.e., U-turns);
316.1945 Stopping, standing or parking prohibited in specified places;
316.195 Additional parking regulations;
316.1995 Driving upon sidewalk or bicycle path; and.
316.2065 Bicycle regulations.

The current versions of these sections are available on the Internet.

Driveways and Access Control: ***AASHTO*** defines driveways as at-grade intersections and states that they should not be located within the functional boundary of at-grade intersections since accidents are disproportionately higher at driveways than at other intersections. The design and operation of driveways are influenced by:

1. Type of adjoining land use;
2. Dimensions of the property;
3. Trip generation characteristics of the site;
4. Design vehicle(s);
5. Type of highway abutting the driveway and
6. Neighboring driveways and driveways on opposite side of roadway.

The following conditions may justify access management features:

1. There is a need to provide an organized movement of traffic within an urban area;
2. Access management is required to provide acceptable capacity and safety and
3. Access management is likely to reduce the chance that a highway or artery will need to be relocated or reconstructed.

Drainage: Drainage is usually more difficult and costly for urban areas. There is greater need to intercept concentrated storm water before it reaches the streets and to remove over-the-curb flow and surface water without interrupting traffic flow or causing a problem for vehicle occupants or pedestrians. Critical factors to take into consideration are:

1. Runoff volumes;
2. Storm water spread calculations;
3. Potential damage to adjacent property by flooding;
4. Number of inlets and underground systems needed;
5. Location of natural areas of water bodies to receive flood water and
6. Problems for bicycles and pedestrians caused by drainage features.

Utilities: It is important to note, by ***Florida Statutes***, that public and privately owned utilities are permitted by the Department to be accommodated within the right-of-way on the SHS. The authoritative reference for utilities is the ***Utility Accommodation Manual (UAM).*** The intent of this manual is to provide direction, policy, criteria, and regulations for the accommodation of utilities within state transportation facility rights-of-way. In addition, ***PPM Volume I, Chapter 5 Utilities***, provides discussion on utility considerations that must be addressed in the design of state highway facilities.

2.5.4 Requirements and Constraints of Traffic Control Devices

Traffic control devices serve the following purposes:

1. Communicating traffic laws and regulations to road users,
2. Warning road users of impending hazards,
3. Delineating vehicle paths within the roadway,
4. Providing information related to destinations, services, etc. and
5. Distributing right of way between competing movements.

The following considerations apply to traffic control devices at intersections.

2.5.4.1 Traffic Control Criteria and Standards

The authoritative reference for traffic control devices is the ***MUTCD***, as described in Chapter 1. ***Florida Statutes*** require conformance to the ***MUTCD*** for traffic control devices installed at all intersections.

The following sections of the ***Florida Statutes*** define additional requirements and constraints for traffic control devices:

316.074 Obedience to and required traffic control devices;
316.0745 Uniform signals and devices;
316.0747 Sale or purchase of traffic control devices by non-governmental entities; prohibitions;
316.075 Traffic control signal devices;
316.0755 Pedestrian control signals;
316.076 Flashing signals;
316.0765 Lane direction control signals and
316.077 Display of unauthorized signs, signals or markings.

Traffic control devices are categorized in three groups: markings, signs and signals. Each of these groups is covered in detail in a separate chapter of the ***MUTCD***. Although not technically a device, an additional form of traffic control is the modern roundabout, which controls traffic through the use of specific geometry, as well as, markings and signs.

2.5.4.2 Pavement Markings

Pavement markings are an important component of intersection design, providing guidance to motorists in terms of lane assignment on approaches, as well as, proper paths for turning movements or unavoidable lane offsets through an intersection. Pavement markings are generally prepared in conjunction with the geometric design of roadways, and must be coordinated with intersection operations.

2.5.4.3 Signs

Signs communicate selected traffic laws, call attention to unexpected conditions, or provide direction to routes and destinations. Signs can be used alone or to supplement pavement markings or traffic signals, and can be critical to intersection operations. Signs should be used judiciously and sparingly to avoid an information overload.

2.5.4.4 Traffic Signals

Traffic signals provide for orderly flow through intersections by alternately assigning right-of-way to particular movements. The required engineering study to justify a traffic signal should address the relative safety and capacity impacts of installation. If a traffic signal is justified, the intersection should be designed in accordance with optimized traffic signal operations. Detailed considerations for signalization are provided in Chapter 4 of this guide.

2.5.4.5 Roundabouts

Roundabouts perform a traffic control function much like signalization in distributing right-of-way, but require less maintenance; no power to operate, no retiming necessary e.g. Roundabouts minimize the number of movements and conflict points within the intersection in order to reduce the possibility of crashes and decrease delay. Detailed considerations for roundabouts are provided in Chapter 7 of this guide.

2.5.5 Objects and Amenities

All other physical roadside features are referred to in this document as objects and amenities. The principal items in this category include lighting and landscaping. Other miscellaneous items of street furniture, mailboxes, etc. fall under this general designation. The most important requirements and constraints associated with objects and amenities include.

A. **Intersection Lighting**: Factors that are important in determining the minimum conditions to justify lighting include:

1. Traffic volume, (motor vehicles, pedestrians and bicycles),
2. Speed,
3. Road use at night,
4. Night accident rate,
5. Road geometrics and
6. General night visibility.

The principal reference governing intersection lighting is "*Roadway Lighting Design Guide,*" published by AASHTO. More details on intersection lighting are presented in Chapter 6.

B. **Landscaping**: According to ***AASHTO***, the philosophy behind landscape development is based on keeping with the character of the highway and its environment. Improvement programs include: (1) preservation of existing vegetation, (2) transplanting of existing vegetation where practical, (3) planting of new vegetation, (4) selective clearing and thinning and (5) regeneration of natural plant species and material. The objectives of these programs are to provide vegetation that will:

1. Improve aesthetics, operations and safety;
2. Lower construction and maintenance costs;
3. Create interest, usefulness and beauty for the pleasure and satisfaction of the traveling public;
4. Mitigate the many nuisances associated with urban traffic and
5. Provide a barrier between motor vehicles and other traffic modes

The authoritative references on landscaping include "*A Guide for Transportation Landscape and Environmental Design"* published by AASHTO and "*The Florida Landscaping Guide"* published by the FDOT. More detailed information on landscaping will be presented later in Chapter 6.

2.6 DIRECTIONAL ROADWAY UTILIZATION

An undivided roadway is a two way street by default. There are, however, conditions that make other directional utilization schemes desirable. ***Florida Statutes, Section 316.088*** permits the assignment of any part of a roadway for travel in one specified direction part or all of a day. The designer should be aware of the advantages and limitation of one-way streets. The main advantages include:

1. Elimination of crashes between vehicles traveling in opposite directions (head-on, left-with-through and sideswipe);
2. Increased capacity;
3. Improved utilization of streets with an odd number of lanes;
4. Simplification of signal phasing through elimination of left-turn conflicts;
5. Improved signal progression and
6. Improved access at cross streets because of more favorable platoon formation.

On the other hand, one-way streets have some limitations of their own, the most significant of which are:

1. Complexity and proliferation of traffic signs;
2. Increased circulation requirements;
3. Terminal treatment of one-way pairs;
4. Loss of public transit curb space and route flexibility;
5. Problems of emergency vehicle routing and signal blockage;
6. Hazards to crossing pedestrians and cyclists resulting from violation of expectancy and
7. Loss of center pedestrian refuge.

2.7 ACCESS MANAGEMENT FOR INTERSECTIONS

Access Management is the practice of controlling the design and placement of access features (such as driveways, side-streets and median openings) in such a way as to enhance the safety and operations of the highway system. Because intersections are a major control of the highway system, it is important to consider the placement and design of driveways, median and median openings especially close to intersections.

Driveways and median openings close to an intersection create a situation where the road user must negotiate conflicts too close to an area that has been designed to manage large volumes of traffic and its own inherent conflicts. Proper driveway placement can also help the business operator because today's traffic queues are so long that traffic exiting driveways may be blocked for long periods of time.

FDOT has developed standards and practices to help in the proper access management treatment near intersections. Chapter 3 contains more information on selected access-management topics, including access classification, driveway design and median design. Roundabouts are an effective control measure in developing an access management plan in that the U-turn movement can preclude the need for downstream left turns.

2.8 DATA REQUIREMENTS FOR INTERSECTION DESIGN

The data requirements will depend on the nature of the design project. The following information will be required for most projects:

1. Approach volumes for each intersection approach, typically 24 hour volumes summarized by 15 minute intervals;
2. Peak hour turning movement counts;
3. Existing geometrics;
4. Pedestrian and bicycle volumes, if applicable;
5. Distances to other intersections;
6. Crash history;
7. Institutional locations: schools, etc.;
8. Posted speed limits along the intersecting roads;
9. Physical and right-of-way features and limitations;
10. Site development features: businesses, driveways, etc.; and,
11. Community considerations: need for parking, landscape, character, etc.

In addition, information on the following items may be needed, depending on specific design project objectives:

1. Anticipated growth based on governing comprehensive plan;
2. Existence of traffic management strategies existing in the area;
3. Types of vehicles using the intersecting roadways;
4. Transit routes along intersecting roadway;
5. Surrounding land use, especially if the proposed design is a community enhancement project;
6. Access to adjacent properties;
7. Compatibility with adjacent intersections;
8. Availability of power and lighting; and,
9. The location of existing above-ground and below-ground utilities.

2.9 TYPICAL INTERSECTION LAYOUTS

Typical layouts for an urban intersection, suburban or rural intersection, and roundabouts are illustrated in ***Figure 2-5*** and ***Figure 2-6***, with the principal components labeled. The intersection design process involves choosing or computing values for the various design parameters. The geometric design aspects of this process will be covered in ***Chapters 3*** and 7. The traffic operational aspects will be covered in ***Chapters 4 and 5***. Note that the design details shown in these figures are intended for purposes of general illustration and should not be interpreted as specific recommendations

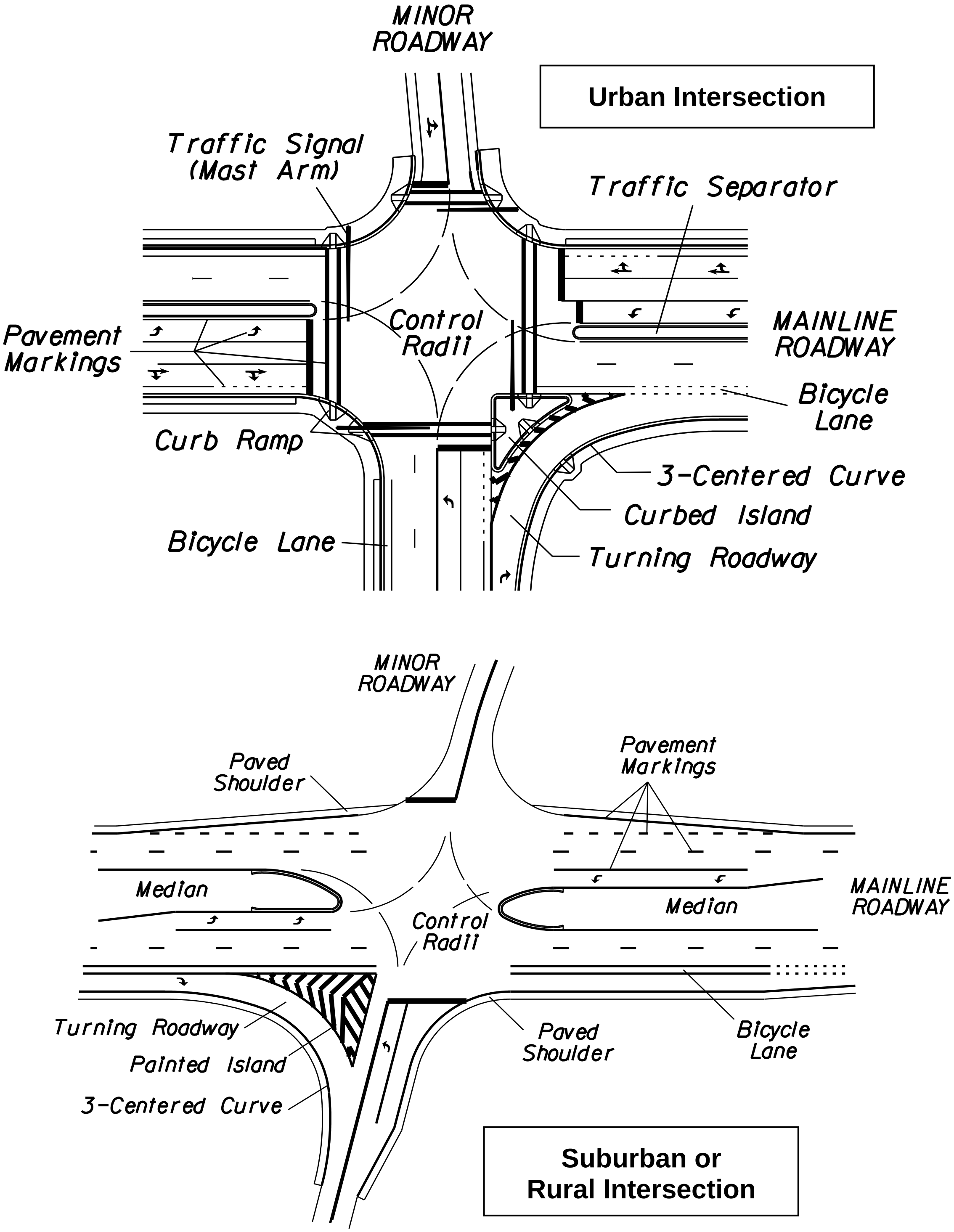

Figure 2-4 Typical Layouts for Urban and Rural Intersections

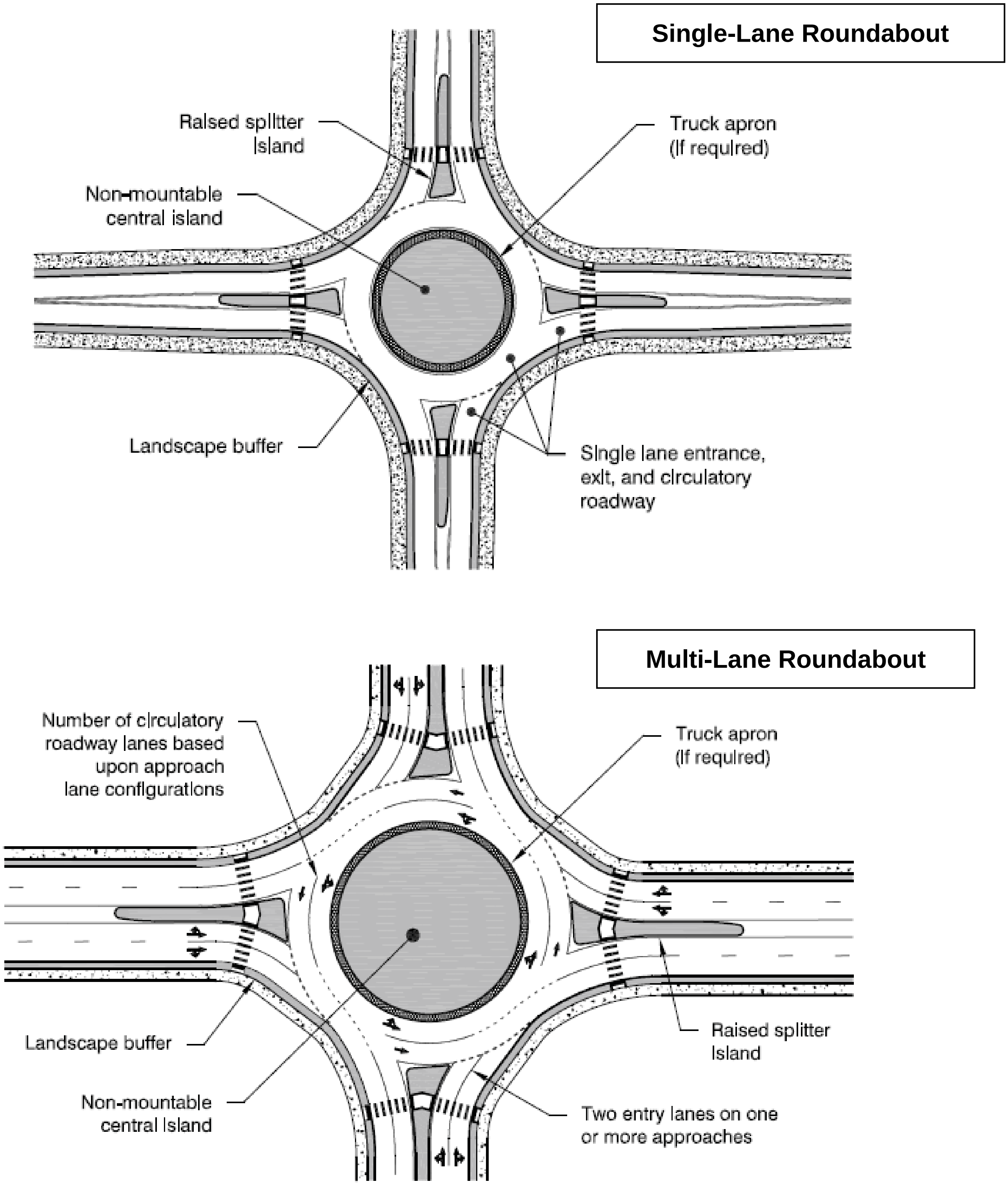

Figure 2-5 Typical Layouts for Single- and Multi-lane Roundabouts

2.10 DETERMINING THE BASIC INTERSECTION CONFIGURATION

The basic intersection configuration is specified in terms of the number of lanes and lane utilization on each approach to the intersection. The basic configuration will be subject to an iterative sequence of changes and refinements as the detailed design procedures described in subsequent chapters of this guide are applied.

The basic configuration begins with the number of through lanes available on the two intersecting streets. Additional lanes will often be necessary to accommodate left and right turns. In some cases a widening of the intersection may be required to accommodate additional through lanes on the approach, with a lane drop on the exit. Multiple left or right turn lanes may be required in addition to channelization for left or right turns. Roundabouts preclude the need for additional turn lanes and exclusive queuing space.

It is important to consider at the outset the effect of the basic intersection configuration on all modes of travel and for all road users. While the provision of adequate capacity and vehicular safety will usually be the principal determinants, the tradeoff between capacity for vehicles and safety for pedestrians and bicyclists must be addressed. Recently developed procedures outlined in the ***HCM*** provide a means for evaluating pedestrian "quality of service", and the ***HSM*** provides procedures for quantifying vehicular, bicycle, and pedestrian safety. In general, any provision that requires modification of the intersection should be justified on the basis of its demonstrated importance to the safety and capacity of the intersection, and mitigation of its adverse effects should be considered.

> Before an intersection is widened, the additional green time pedestrians need to cross the widened roadways should be considered to ensure that it would not exceed the green time saved through improved vehicular flow.
>
> ***{MUTCD}***

Chapter 3

Geometric Design

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3 GEOMETRIC DESIGN

Geometric design involves the proportioning of the visible elements of highway facilities. It includes the design of horizontal alignment, vertical alignment and cross section elements such as shoulder, median, curb, barrier, sidewalk, etc. At intersections, these elements and their configurations are influenced by traffic demands and the proper accommodations of traffic control devices.

Although the design of an intersection may be influenced by constraints unique to its particular location or situation, it conforms generally to the following design principles:

1. The design of intersections along a given street or highway should be as consistent as possible.
2. The layout of the intersection should be as simple as is practicable.
3. The design of all intersection elements should be consistent with the approach design speeds.
4. The approach roadways should be free from steep grades or sharp horizontal or vertical curves.
5. Conventional intersections should be as close to right angle as practical, or a roundabout should be considered.
6. Sight distance should be sufficient for all movements approaching and proceeding through the intersection.
7. The intersection layout should encourage smooth flow and discourage wrong-way movements.
8. Auxiliary turn lanes should be provided in conventional intersections on high-speed and/or high-volume facilities.
9. Acceleration lanes are desirable for entrance maneuvers onto high-speed facilities.
10. Design must give special attention to the provision of safe roadside clear zones and horizontal clearance.
11. The intersection arrangement should not require sudden and/or complex decisions.
12. The layout of an intersection should be clear and understandable.
13. Bicycle and pedestrian traffic in urban areas is an integral component of the demand traffic volume and must be accommodated.
14. Advance guidance and/or lane assignment should be provided on the approaches to intersections.

This chapter presents criteria for the design of at-grade intersections for new and major reconstruction projects on the Florida State Highway System (SHS). Where a non-SHS roadway is connected to a SHS roadway, the geometric design of the functional area of the non-SHS roadway must meet the criteria set forth in this chapter. The concept of functional area was defined in Chapter 2.

3.1 SIGNIFICANT REFERENCES

In addition to the authoritative references described in Chapter 1, the following reference documents govern the geometric design aspects for intersections in Florida:

1. Intersection Channelization Design Guide. National Cooperative Highway Research Program Report (NCHRP) 279. Transportation Research Board, National Research Council. November 1985.
2. Older Driver Highway Design Handbook. U.S. Department of Transportation, Federal Highway Administration (FHWA). October 2001.
3. Transportation and Land Development. Vergil G. Stover and Frank J. Koepke. Institute of Transportation Engineers. 1988.

3.2 INTERSECTION TYPES

Intersection types can be categorized by intersection basic type, functional classification, control type, area type, or a combination of these classifiers, depending on the element of design.

3.2.1 Basic Type

An at-grade intersection can be three-leg (T or Y), four-leg, multi-leg or circular. In intersection design, the type of intersection is established first and then an appropriate geometric plan is developed, reflecting suitable design and operational criteria within the physical constraints.

Roundabouts are circular intersections with yield control on all approaches (entries). Currently, FDOT is recommending a limit of two lanes within the circulatory roadway, although additional lanes may be considered in the case of spiral or “Turbo” roundabouts. Spiral or Turbo roundabouts are typically used in cases where more than two lanes are required to accommodate certain movements.

3.2.2 Functional Classification

"Functional classification" means the assignment of roads into systems according to the character of service they provide in relation to the total road network using procedures developed by the Federal Highway Administration. Basic functional categories include arterial roads, collector roads, and local roads which may be subdivided into principal, major, or minor levels. Those levels may be additionally divided into rural and urban categories. [***Florida Statutes, 334.03 (10)***]

Facilities on the State Road System are now classified according to the following categories:

Functional Classification Codes:

1 = Interstate

2 = Other Freeways and Expressways

3 = Other Principal Arterial

4 = Minor Arterial

5 = Major Collector

6 = Minor Collector

7 = Local

Functional classifications and the standards required by them are predetermined controls over which the designer has little choice. The standards are minimum values and values above the minimum should be used where possible and practical [***PPM***]. Decisions regarding specific design features should be evaluated using procedures identified in the Highway Safety Manual.

3.2.3 Intersection Control Modes

At-grade intersections on the SHS are typically controlled by stop signs (i.e., stop controlled), roundabouts, or traffic signals (i.e., signalized). Certain channelized movements at intersections and interchanges, and all approaches to roundabouts are yield controlled. The type of intersection control has a direct effect on a number of geometric design features, including lane assignments, sight distance and storage length of auxiliary lanes.

3.2.4 Area Type

Area type is typically stratified as urban, suburban, or rural. Each of these area types has fundamentally different characteristics with regard to development and types of land use, density of street and highway network, nature of travel patterns and ways in which these elements are related. Consequently, the intersection design considerations and requirements for each of these areas vary.

In certain areas, the Department may designate specific projects or segments of projects to incorporate Transportation Design for Livable Communities (TDLC) features. See the ***PPM,*** Volume I, Chapter 21 for information on TDLC considerations, features and requirements for these types of projects.

3.2.5 Access Management

Access management affects intersection design with relation to the location of access points, median openings, and their vicinities to intersections. Some driveways are of such scale as to be considered intersections in their own right. Rule 14-97 of the Florida Administrative Code establishes the seven classifications for state highways and the criteria and procedures for assigning these classifications to specific roads. These classifications contain separation standards for access features. Essentially, the Department of Transportation determines which roads are the most critical to providing high speed, high volume traffic, and assign the highest standards these facilities.

Roundabouts are especially useful in access management strategies in that the U-turn movement affords the opportunity for restrictive medians.

Designers should consult the Planning Office's Access Management website for specific guidance regarding access management:

http://www.dot.state.fl.us/planning/systems/sm/accman/

3.3 DESIGN SPEEDS

Design speed is a principal design control that regulates the selection of many of the project standards and criteria used to design a roadway project. The mainline design speed will influence the design elements of the intersection such as selection of control mode, location and design of islands, taper lengths, and sight distance requirements.

Vehicles turning at intersections designed for minimum-radius turns have to operate at speeds of less than 10 mph. While it may be desirable and at times feasible to design for turning vehicles operating at higher speeds, it is often necessary for safety and economy to use lower turning speeds at most at-grade intersections. The speeds for which intersection curves should be designed depend on approaching vehicle speeds, design vehicle, type of intersection, control mode, pedestrian volume and through and turning volumes.

3.4 DESIGN VEHICLES

A design vehicle is a selected motor vehicle with the weight, dimensions and operating characteristics used to establish highway design controls for accommodating vehicles of a designated class. For purposes of geometric design, each design vehicle has larger physical dimensions and a larger minimum turning radius than those of almost all vehicles in its class.

The selected design vehicle significantly affects intersection design, including horizontal and vertical alignments, lane widths, inscribed circle diameter, turning radii, lane assignments, intersection sight distance, storage length of auxiliary lanes, and acceleration and deceleration lengths on auxiliary lanes.

The ***AASHTO Green Book*** includes a variety of design vehicles. The dimensions of these vehicles are presented in ***Table 3-3***. The choice of design vehicle is influenced by the functional classification of a roadway and by the proportions of the various types and sizes of vehicles expected to use the facility. On SHS facilities, to accommodate truck traffic, one of the semi-trailer vehicles should be considered in design. In urban areas that are highly built-up, intersections may be designed to provide fully for passenger vehicles but require the larger vehicles to swing wide upon turning. It should be noted that the WB-62 design vehicle modified with a 53 foot trailer and a 41 foot KCRT (kingpin to the center of the rear tandem axle) distance accommodates the maximum dimensions allowed by ***FS 316.515***. This semitrailer, known as the "WB-62FL" is depicted in ***Figure 3-1*** and should be used for designing turning roadways in Florida. A larger design vehicle may be used if special conditions exist. See Section 3.13.1 and ***Table 3-11*** for information regarding the different turn radii that may be used to accommodate various design vehicles.

Computer generated turning templates, allows the roadway designer to select a design vehicle and simulate the expected turning path. Any proposed intersection design/layout should be tested with such a computer program, or alternatively with turning path templates, to make sure it can handle the turning movements of the design vehicle. ***Figure 3-1*** shows an example of the swept path template for testing the turn radius adequacy for a WB-62FL design vehicle.

Table 3-1 Dimensions, in Feet, of Design Vehicles [*AASHTO, 2011*]

Design Vehicle Type	Symbol	Overall			Overhang		WB_1	WB_2	S	T	WB_3	WB_4	Typical Kingpin to Center of Rear Tandem Axle
		Height	Width	Length	Front	Rear							
Passenger Car	P	4.3	7.0	19.0	3.0	5.0	11.0	-	-	-	-	-	-
Single-Unit Truck	SU-30	11.0-13.5	8.0	30.0	4.0	6.0	20.0	-	-	-	-	-	-
Single-Unit Truck (three-axle)	SU-40	11.0-13.5	8.0	39.5	4.0	10.5	25.0	-	-	-	-	-	-
Buses													
Intercity Bus (Motor Coaches)	BUS-40	12.0	8.5	40.5	6.3	9.0[a]	25.3	-	-	-	-	-	-
City Transit Bus	BUS-45	12.0	8.5	45.5	6.2	9.0[a]	28.5	-	-	-	-	-	-
Conventional School Bus (65 pass.)	S-BUS 36	10.5	8.0	35.8	2.5	12.0	21.3	-	-	-	-	-	-
Large School Bus (84 pass.)	S-BUS 40	10.5	8.0	40.0	7.0	13.0	20.0	-	-	-	-	-	-
Articulated Bus	A-BUS	11.0	8.5	60.0	8.6	10.0	22.0	19.4	6.2[b]	13.2[b]	-	-	-
Combination Trucks													
Intermediate Semitrailer	WB-40	13.5	8.0	45.5	3.0	4.5[a]	12.5	25.5	-	-	-	-	25.5
Interstate Semitrailer	WB-62**	13.5	8.5	69.0	4.0	4.5[a]	19.5	41.0	-	-	-	-	41.0
Interstate Semitrailer	WB-67**	13.5	8.5	73.5	4.0	4.5[a]	19.5	45.5	-	-	-	-	45.5
"Double-Bottom" Semitrailer/Trailer	WB-67D	13.5	8.5	72.3	2.3	3.0	11.0	23.0	3.0[c]	7.0[c]	22.5	-	23.0
Rocky Mountain Double-Semitrailer/Trailer	WB-92D	13.5	8.5	97.3	2.3	3.0	17.5	40.0	4.5	7.0	22.5		40.5
Triple-Semitrailer/Trailers	WB-100T	13.5	8.5	104.8	2.3	3.0	11.0	22.5	3.0[d]	7.0[d]	22.5	22.5	23.0
Turnpike Double-Semitrailer/Trailer	WB-109D*	13.5	8.5	114.0	2.3	4.5[a]	12.2	40.0	4.5[a]	10.0[e]	40.0	-	40.5
Recreational Vehicles													
Motor Home	MH	12.0	8.0	30.0	4.0	6.0	20.0	-	-	-	-	-	-
Car and Camper Trailer	P/T	10.0	8.0	53.0	4.0	8.0	20.0	-	5.0	17.7	-	-	-
Car and Boat Trailer	P/B	-	8.0	42.0	3.0	8.0	11.0	-	5.0	15.0	-	-	-
Motor Home and Boat Trailer	MH/B	12.0	8.0	53.0	4.0	8.0	20.0	-	6.0	15.0	-	-	-

* Design Vehicle with 48.0-ft trailer as adopted in 1982 Surface Transportation Assistance Act (STAA).

** Design Vehicle with 53.0-ft trailer as grandfathered in with 1982 Surface Transportation Assistance Act.

[a] This is the length of the overhang from the back axle of the tandem axle assembly.

[b] Combined dimension is 19.4 ft and articulating section is 4.0 ft wide.

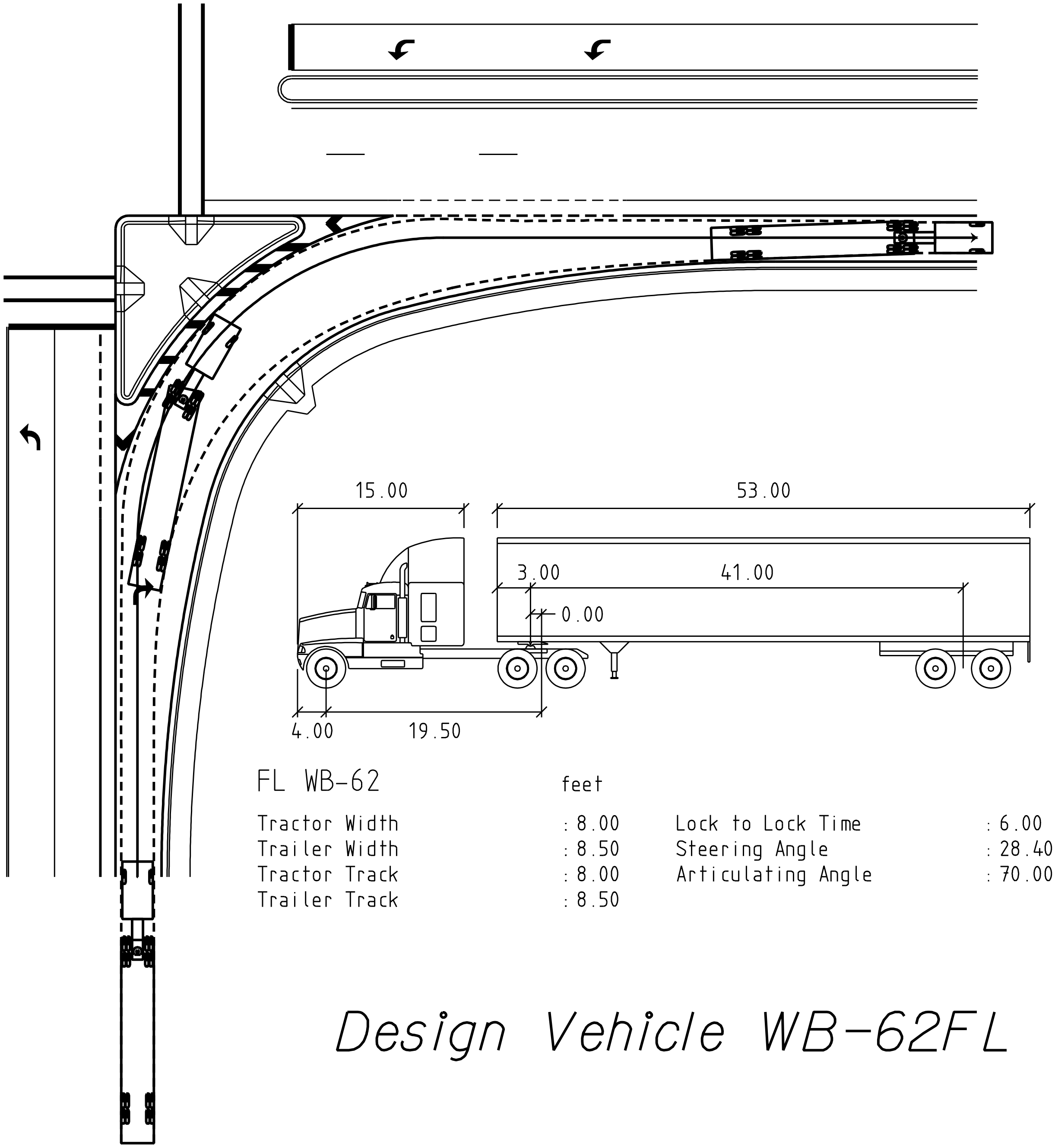
[c] Combined dimension is typically 10.0 ft.

[d] Combined dimension is typically 10.0 ft.

[e] Combined dimension is typically 12.5 ft.

- WB_1, WB_2, WB_3, and WB_4 are the effective vehicle wheelbases, or distances between axle groups, starting at the front and working toward the back of each unit.
- S is the distance from the rear effective axle to the hitch point or point of articulation.
- T is the distance from the hitch point or point of articulation measured back to the left of the next axle or the left of the tandem axle assembly.

Figure 3-1 Swept Path Turn Radius Example



3.5 PEDESTRIAN TRAFFIC

Pedestrian traffic can be an integral component of the demand traffic stream and must be incorporated into the original intersection design. All new or major reconstruction projects should be designed with the assumption that pedestrians and bicyclists will use them. The ***Plans Preparation Manua***l, Chapter 8, gives specific direction on required bicycle and pedestrian facilities; Decisions on additional pedestrian and bicycle facilities and/or features shall be determined with input from the District Pedestrian/Bicycle Coordinators and District **Americans with Disabilities Act (ADA)** Coordinators [PPM].

Return radii at an intersection must balance the needs of the pedestrian and the design vehicle. Larger radii are needed to accommodate a vehicle's turning ability while smaller radii are needed to minimize the crossing distance for pedestrians. In cases where large radii are unavoidable, consideration should be given to incorporating channelization islands for pedestrian refuge. In urban areas, where a parking lane is present, curb extensions may be used to minimize the crossing distance [PPM].

Pedestrian facilities must be designed in accordance with **ADA** to accommodate those who are visually challenged or depend on wheelchairs and other devices for mobility. Curb ramps should be constructed at locations where marked crosswalks intersect the raised sidewalk. Section 3.10.4 contains additional information on the implementation of curb ramps.

For information on signal timing considerations for pedestrians, see Chapter 4 of this guide.

3.6 BICYCLE TRAFFIC

When on-street bicycle lanes and/or off-street shared use paths enter an intersection, these facilities must be continued through the intersection. In addition, even in locations where there are no bicycle facilities, the inclusion of bicycle lanes on intersection improvement projects must be considered.

Bicycle lanes may be either designated or undesignated. Designated bicycle lanes are marked with bicycle lane signs and special pavement markings. Undesignated bicycle lanes are separated from traffic lanes by edge stripes, but do not have bicycle lane signs and special pavement markings. The decision to use designated bicycle lanes versus undesignated depends on the expected use, (e.g., the continuity of the bicycle route, the presence of logical route termini for bicyclists, etc.).

Bicycle lane width requirements are provided in ***Table 3-4***. On roadways with flush shoulders, the FDOT standard 5 feet paved shoulder provides for a bicycle lane that may be designated or undesignated. On curb and gutter roadways, a 4 feet width measured from the lip of the gutter is required. This provides for a 5.5 feet width to the face of curb when FDOT Type F curb and gutter is used. The 1.5 feet gutter width should not be considered as part of the rideable surface area, but this width provides useable clearance to the curb face. Where parking is present, the bike lane should be placed between the parking lane and the travel lane and have a minimum width of 5 feet.

At intersections with right turn lanes, the bicycle lane should continue adjacent to the through lane between the through lane and the right turn lane and should be 5 feet in width, 4 feet minimum. The suburban or rural intersection shown in the previous chapter (***Figure 2-4***) provides one example of this treatment. Standard drawings for various bicycle lane configurations are provided in the ***Design Standards, Index 17346*** and ***17347***.

At roundabouts, bicycle lanes should be terminated at bypass ramps to access the sidewalk prior to entering the circulatory roadway. Bicycle riders have the option to "command the lane" and travel through the roundabout in the circulatory roadway, or to divert onto the sidewalk and cross the intersection along pedestrian paths (riders must yield to pedestrians).

Additional information on bicycle facilities is provided in the ***PPM,*** Volume I, Chapter 8. For information on signing and marking requirements, see Chapter 5 of this guide.

3.7 HORIZONTAL ALIGNMENT

3.7.1 Intersection Angle

The angle of intersection of two highways can greatly influence the intersection's safety and operational characteristics. Both individual vehicle operations and the nature of vehicle/vehicle conflicts are affected by angle of intersection. With the exception of roundabouts, acute "skew" intersection angles produce large open pavement areas within the intersection. Such intersections are not only more costly to build and maintain, but are operationally undesirable for the following reasons:

1. Vehicles crossing the intersection are exposed for a longer time to conflicts from crossing traffic. This may be particularly critical at STOP-controlled approaches on high-speed highways.
2. The road user's sight angle to one of the crossing legs becomes more restricted. This increases the difficulty of perceiving safe crossing gaps.
3. Pedestrians and bicyclists are subjected to longer times of exposure to conflicting vehicles.
4. Vehicular movements are more difficult because of the skew. Accommodation of large truck turns may necessitate additional pavement and channelization not otherwise called for. The greater open pavement heightens the opportunity for vehicles to wander out of the proper paths [***NCHRP 279***].

Approaching roadways should intersect at right angles where practical. Angles less than 90 degrees, but greater than 75 degrees, should be maintained normally. Angles as low as 60 degrees may be acceptable where costly or severe constraints occur [***NCHRP 279***].

Due to the large population of older road users in the State of Florida, however, angles less than 75 degrees are generally not recommended [FHWA].

Where severe skew angles exist, the need to consider improvements should be assessed, with primary emphasis given to examination of crash rates and patterns. A high incidence of right-angle crashes, particularly involving vehicles approaching from the acute angles, may be evidence of a problem attributable to the skew [***NCHRP 279***]. A modern roundabout may be used to eliminate right-angle crashes with corresponding reductions to the severity of crashes. Reconfigurations such as those shown in ***Figure 3-2*** should be considered whenever feasible.

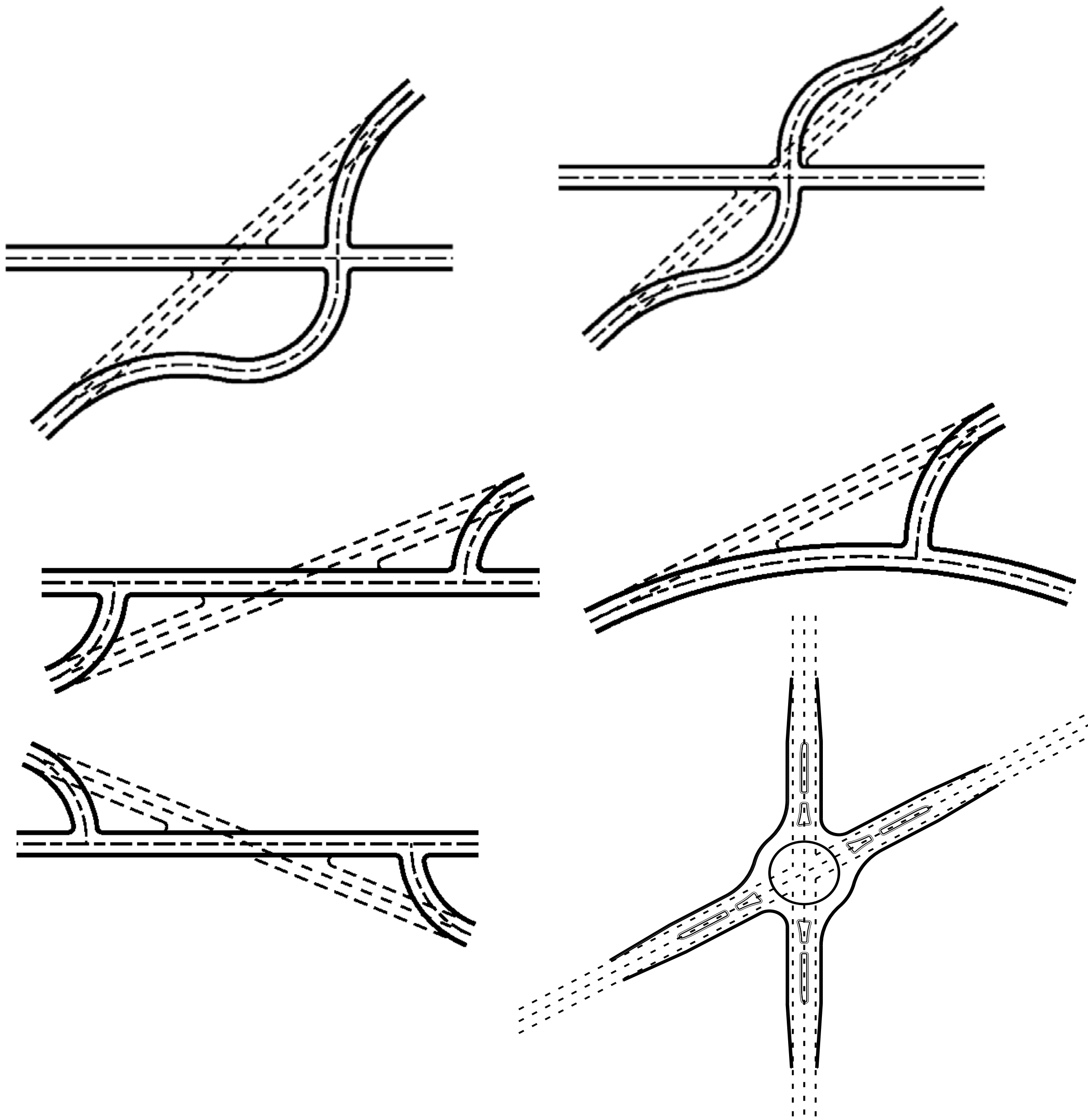


Figure 3-2 Intersection Reconfigurations

3.7.2 Tapers

Tapers may be used through or near an intersection. The taper length, as discussed in this section, is generally based on the following equations:

1. Under merging conditions:
   a. For design speeds equal to 40 mph or less: $L = (W*S^2)/60$
   b. For design speeds equal to or greater than 45 mph: $L = W*S$

   Where: L = Taper length (feet)
   W = Width of offset (feet)
   S = Design speed (mph)
2. Under non-merging conditions, the taper length is equal to L/2.

Transitions

The addition or deletion of through traffic lanes should be undertaken on tangent sections of roadways. Approach taper lengths for auxiliary lanes are given in Section 3.12.3. The termination of lanes, including auxiliary lanes, should meet the general requirements for merging lanes. More information on add or drop lane transitions, including details for various scenarios, can be found in the ***Design Standards, Index 526***.

Lane Shift

A lane shift through an intersection, should meet the general requirements for non-merging lanes. Short curves may be desirable at each end, and pavement markings should be used through the intersection to provide positive guidance to the motorist. The shifting taper length is controlled by the size of the intersection and deflection angle (see ***Figure 3-3***). Lane shifts are undesirable due to potential sight distance restrictions for pavement markings on the far side of intersections.

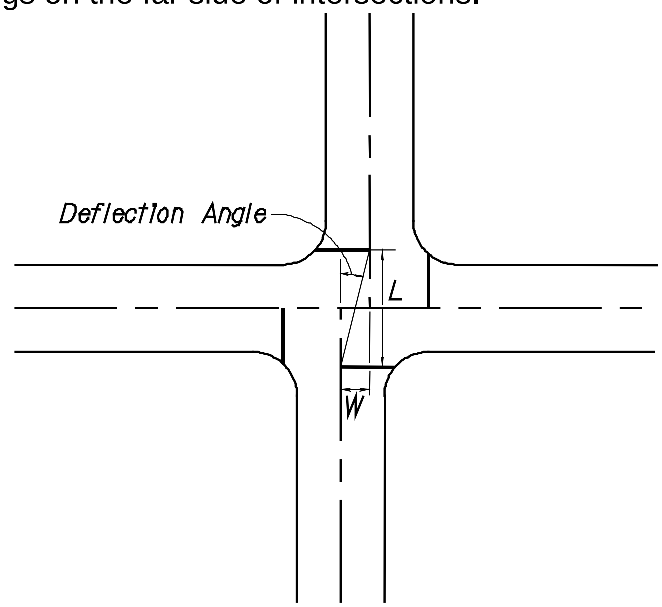


Figure 3-3 Transition for Shifted Lane

Deflection

Redirection or deflection of through lanes through an intersection should meet the general requirements for non-merging lanes. The maximum allowable deflection angle without a curve applied may be derived from taper length equations by using one lane width as the width of offset (W) (see ***Figure 3-4***). Criteria for deflection with a curve can be found in the ***PPM***, Volume I, Chapter 2.

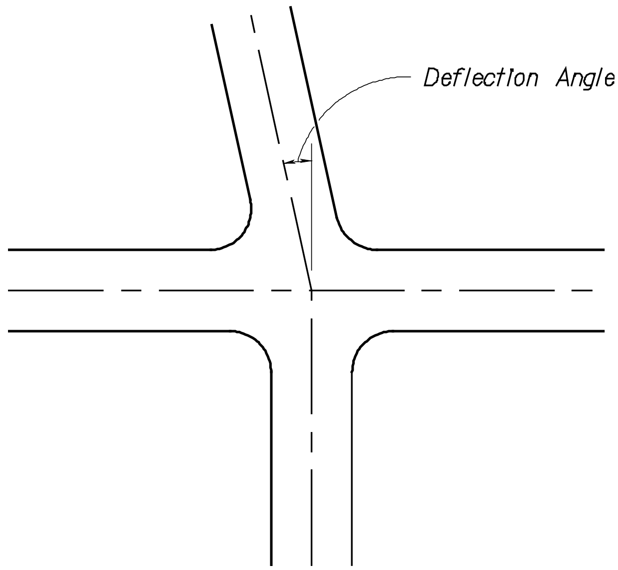


Figure 3-4 Transition for Lane Deflection

3.7.3 Auxiliary Lanes

Auxiliary lanes may be used for certain movements at intersections to improve capacity and operations. Auxiliary lanes can be signal, stop, yield controlled, or free-flowing, where appropriate receiving lanes are present on the intersecting roadway.

3.8 VERTICAL ALIGNMENT

3.8.1 Grade Considerations

The profile grade line defines the vertical alignment for roadway and bridge construction. As with other design elements, the characteristics of vertical alignment are influenced greatly by basic controls related to design speed, traffic volumes, and functional classification, drainage and terrain conditions. Within these basic controls, several general criteria must be considered [PPM], including minimum and maximum grades, vertical curvature, maximum change in grade without vertical curves, vertical clearance and design high water. The ***PPM*** provides specific values for these criteria.

As a rule, the vertical alignment and grades are subject to greater constraints at or near intersections than on the open road. Design for their combination at or near intersections should produce traffic lanes that are clearly visible to drivers at all times and clearly understandable for any desired direction of travel, free from sudden appearance of potential conflicts and consistent in design with the portions of the highway just traveled [AASHTO 11].

Combinations of grade lines that make vehicle control difficult should be avoided at intersections. Substantial grade changes should be avoided at intersections. Adequate sight distance should be provided along both intersecting roads and across their included corners, even where one or both intersecting roads are on vertical curves. The gradients of intersecting roads should be as flat as practical on those sections that are to be used for storage of stopped vehicles.

Most drivers are unable to judge the increase and decrease in stopping or accelerating distance that is necessary because of steep grades; grades in excess of 3% should be avoided on intersecting roads in the vicinity of the intersection. Where conditions make such designs impractical, grades should not exceed 6%.

The profile grade lines and cross sections on the intersection legs should be adjusted for a considered distance back from the intersection to provide a smooth junction and proper drainage. Normally, the grade line of the major road should be carried through the intersection and that of the minor road should be adjusted to it. This design involves a transition in the crown of the minor road to an inclined cross section at its junction with the major road, as demonstrated in ***Figure 3-5***. The break in the cross street profile at the center of the intersection should be accomplished with a vertical curve wherever possible.

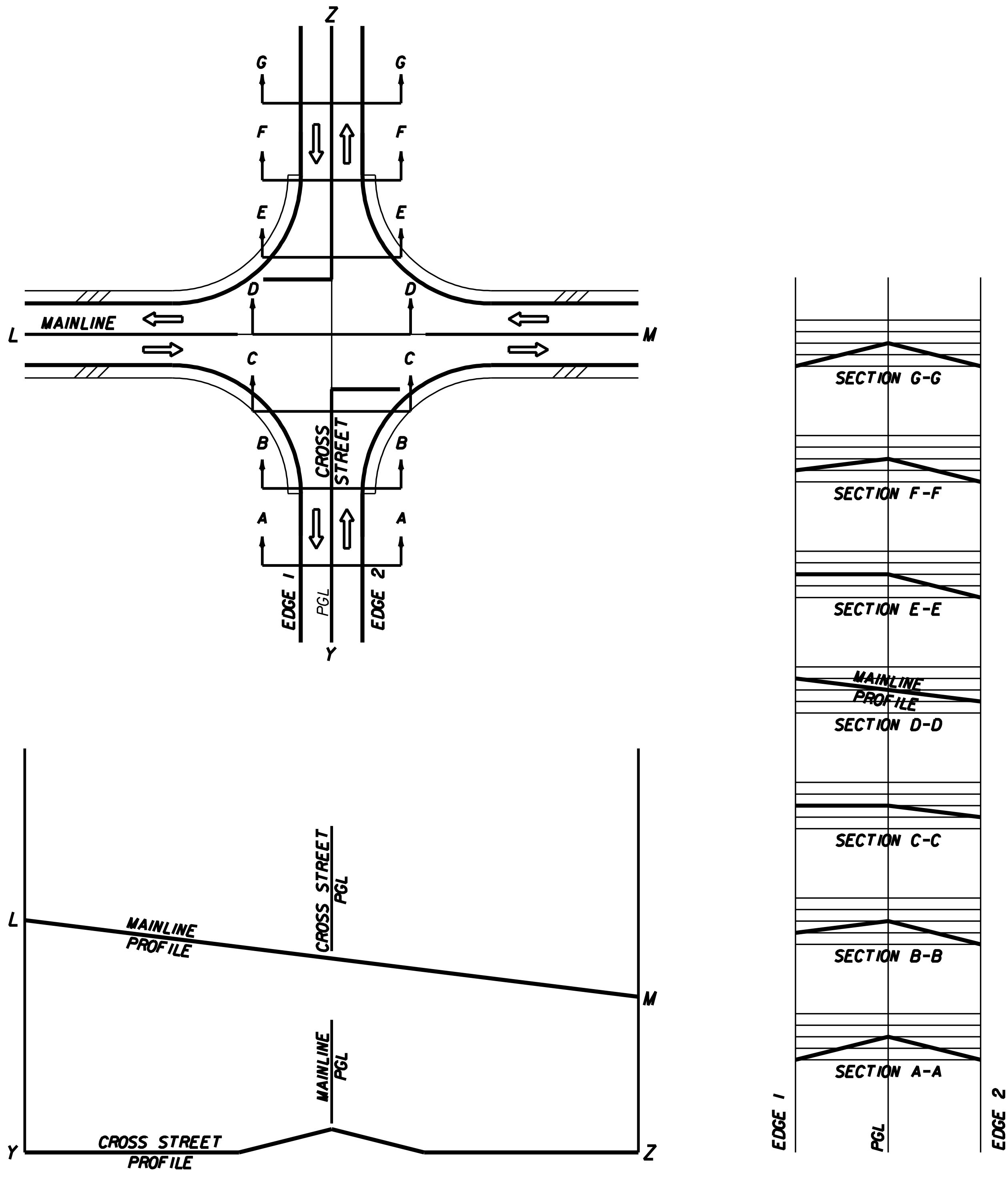


Figure 3-5 Cross Street Intersection Transition

3.8.2 Special Profiles

To ensure a safe, efficient, well drained and smooth roadway system, the profiles of some roadway elements requiring special analysis must be provided. These elements include pavement edges or gutter flow line at street intersections, profile grade line, intersection plateau, curb returns and roadway sections requiring special superelevation details. The special profiles shall include details at close intervals and at a scale large enough to clearly identify all construction details of these elements [***PPM***].

3.8.3 Plateauing

The profile of the major highway generally takes precedence over the minor cross street. This results in a hump for the cross street profile which is particularly undesirable for signalized intersections where the cross street traffic may enter the intersections without stopping. In some instances the designer may determine that the cross street should receive the same profile considerations as the major highway due to similar traffic demands. To provide this "equal treatment", with respect to profile, a technique commonly known as intersection plateauing is applied. Plateauing refers to flattening of the intersection and the transition of both roadway profiles and cross slopes on the intersection approaches.

Guidelines for intersection plateauing are given as follows:

1. All signalized intersections should be considered. However, the following may be used for determining the level of importance:

Type of Signalized Intersection	Level of Need for Plateauing
Multilane highway and Multilane highway	High
Multilane highway and Two-lane highway	Medium
Two-lane highway and Two-lane highway	Low

2. The profile combination selected and the cross section generated as a result should be sufficient to provide a smooth connection and proper drainage. On curb and gutter sections, care must be taken to ensure that the runoff-spread restrictions are not compromised.

3. Transition slope rates for intersection approaches should conform to the values provided in the following table. However, the cross slope transition length shall have a minimum value of 50 feet for design speeds under 40 mph and 75 feet for design speeds of 40 mph or greater.

Speed (mph)	Slope Ratio
30	1:100
40	1:125
45-50	1:150
55-60	1:170
65-70	1:190

An example of a plateaued intersection is presented in ***Figure 3-6***.

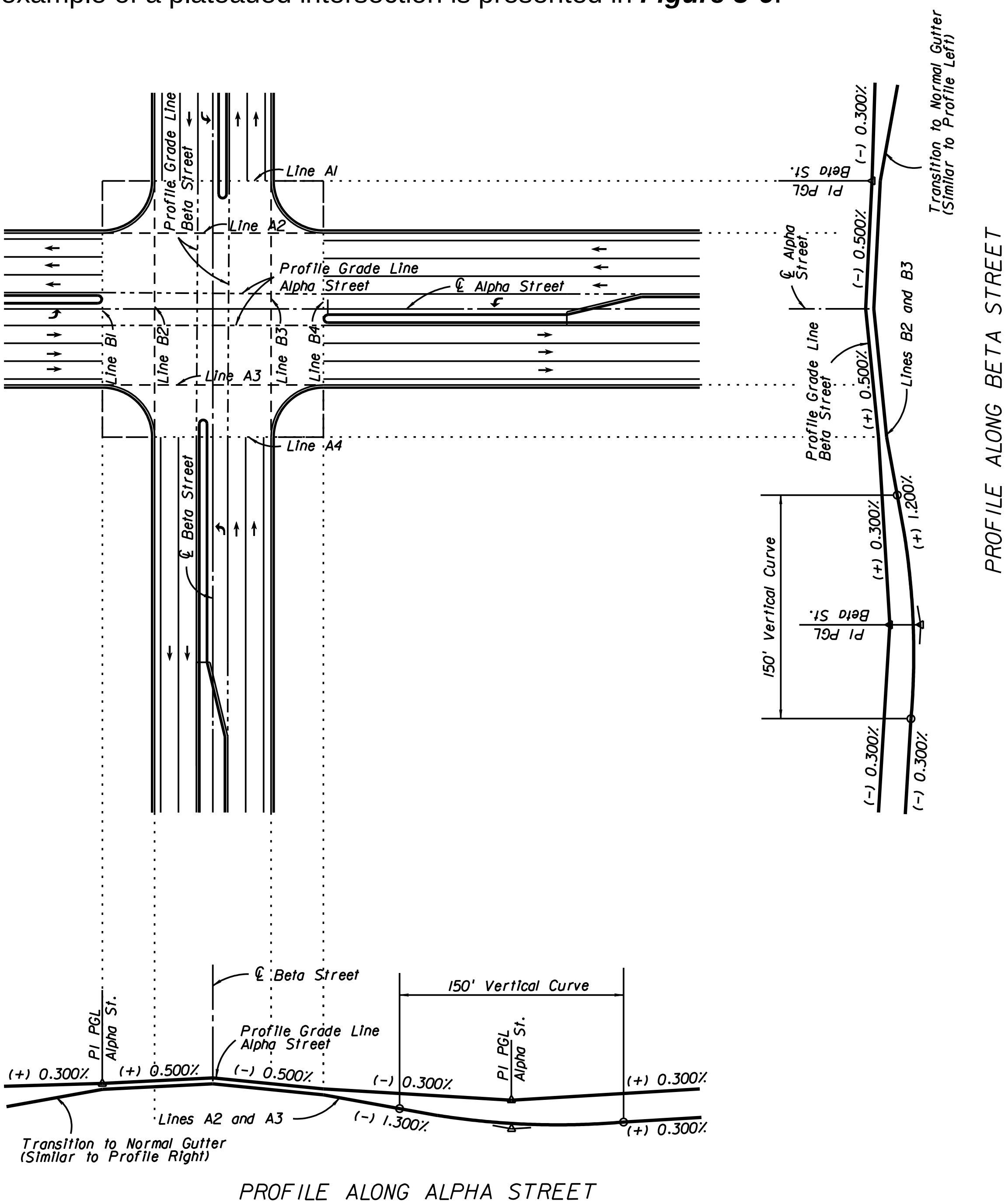


Figure 3-6 Example of Plateaued Intersection

3.9 CROSS SECTION ELEMENTS

3.9.1 Lane Widths

Criteria for the widths of through and turn lanes are given by highway and area types, as given in ***Table 2.1.1***, ***Volume 1***, ***Chapter 2*** of the ***PPM***.

Collector-distributor lanes and auxiliary lanes for speed change, turning, storage for turning, weaving and other purposes supplementary to through-traffic movement should be of the same width as the through lanes [***Table 2.1.2***, ***Volume 1***, ***Chapter 2*** of the ***PPM***]. Auxiliary lanes are discussed further in Section 3.12, turning roadway widths are covered in Section 3.13.4 and widths for other lane types are provided in the ***PPM***.

3.9.2 Median Widths

Median width is the distance between the inside (median) edges of the travel lanes of each roadway. Criteria for median widths of arterials and collectors are given in ***Table 3-2*** [***PPM***].

Wherever possible, a newly designed divided highway should have a median width that can accommodate normal left-turns and passenger car U-turns by using a sufficient intersection design and a median storage lane that will protect and store the design-hour turning volume (see ***Figure 3-7***). If adequate median width does not exist for accommodating U-turns, then adding extra pavement width, through use of a taper or on the shoulder should be considered. Basic median functions and their required width are provided in ***Table 3-3***.

Table 3-2 Median Widths for Arterials and Collectors [*PPM*]

MEDIAN WIDTHS (FEET)	
TYPE FACILITY	**WIDTH**
FREEWAYS	
Interstate, Without Barrier	64 [1]
Other Freeways, Without Barrier	---
Design Speed ≥ 60 mph	60
Design Speed < 60 mph	40
All, With Barrier, All Design Speeds	26 [2]
ARTERIAL AND COLLECTORS	
Design Speed > 45 mph	40
Design Speed ≤ 45 mph	22 [3]
Paved And Painted For Left Turns	12 [4]

Median width is the distance between the inside (median) edge of the travel lane of each roadway.

1 88 ft. when future lanes planned.

2. Based on 2 ft. median barrier and 12 ft. shoulder.

3. On reconstruction projects where existing curb locations are fixed due to severe right of way constraints, the minimum width may be reduced to 19.5 ft. for design speeds = 45 mph, and to 15.5 ft. for design speeds ≤ 40 mph.

4. Restricted to 5-lane sections with design speeds ≤ 40 mph. On reconstruction projects where existing curb locations are fixed due to severe right of way constraints, the minimum width may be reduced to 10 ft. These flush medians are to include sections of raised or restrictive median for pedestrian refuge and to conform to ***Section 2.2.2*** of this volume and the Access Management Rules.

Figure 3-7 Example Median Design for U-Turn Accommodation

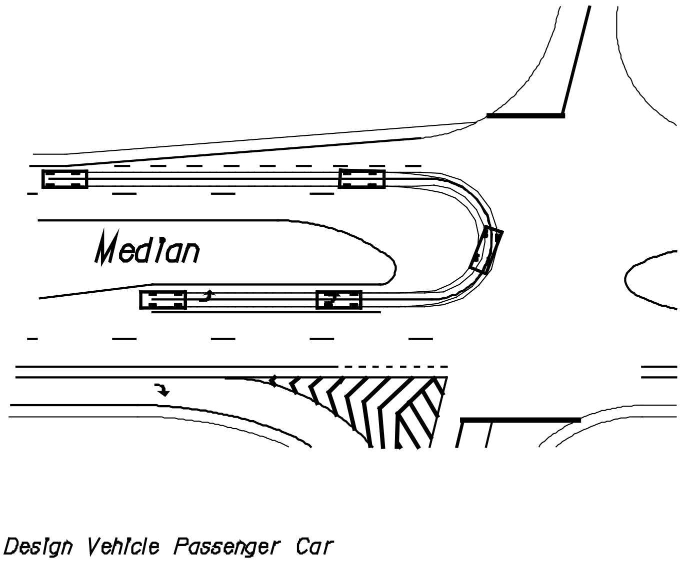


Table 3-3 Basic Median Functions and their Required Width

Function	Width (feet)
Separation of opposing traffic	4
Provision for pedestrian refuge	6
Provision for storage of left-turning vehicles	12_1
Provision for protection of vehicles crossing through lanes	22_2
Provision for U-turns, left turn lane to outside lanes	30
Provision for Dual Left Turn Lanes and U Turns	42

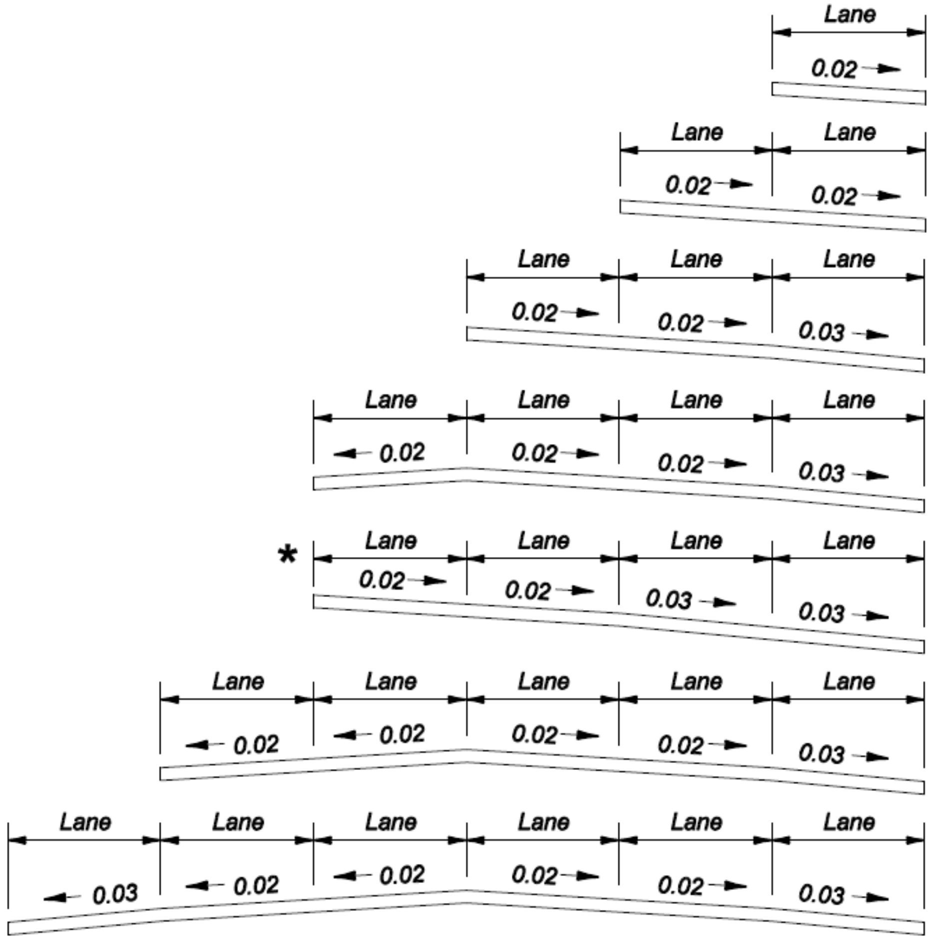
1. See ***Table 3-2*** for conditions.
2. Based on the passenger car (P) design vehicle.

3.9.3 Cross Slopes

The algebraic difference in cross slope between adjacent through lanes should not exceed 0.04. Cross slopes on bridges shall be on a uniform, straight-line rate, typically 0.02, in each traffic direction, with no break in slope. The straight-line slope shall be applied uniformly over all travel lanes and required shoulders in each direction of travel. Bridges with one-way traffic shall have one, uniform cross slope, while bridges with two-way traffic may be designed with a crowned bridge deck section [***PPM***]. See Section 3.13.5 for cross slope requirements for turning roadways.

Figure 3-8 illustrates standard pavement cross slope configurations.

Figure 3-8 Standard Pavement Cross Slopes for Through Lanes [*PPM*]



All Travel Lanes One Direction

These sections show only the standard slopes for adjoining travel lanes; they do not prescribe needed lanes, lane usage or typical section requirements other than lane slope. These slopes are not applicable to parabolic crowns.

Maximum pavement cross slopes on tangent sections are:
- 0.04 for design speeds of 45 mph or less.
- 0.03 for design speeds greater than 45 mph.

The change in cross slope between adjacent through lanes shall not exceed 0.04.

Slopes on multi-purpose lanes may be 0.03 to 0.05. Portions of multi-purpose lanes that are reserved for parking and access isles for the physically disabled shall have cross slopes not exceeding 1:50 (0.02) in all directions.

*NOTE: Four travel lanes may be sloped in one direction for curb and gutter sections only.

3.9.4 Shoulders

Criteria for shoulder widths and slopes for facilities on divided arterials, undivided arterials and collectors are provided in the ***PPM***, ***Volume I***, ***Chapter 2***. It is the FDOT's policy that 5 feet paved outside shoulders are required on all new construction, reconstruction and lane addition projects for all highways except freeways.

3.9.5 Curbs

Curbs are generally designed with a gutter to form a combination curb and gutter section. They are used to provide greater use of the available width, discourage vehicles from leaving the roadway, provide drainage control and to improve delineation of the roadway. Curbs are used extensively on all types of urban highways with design speeds less than 50 mph. In the interest of safety, curbs should be omitted on rural highways when the same objectives can be attained by other acceptable means. The two most commonly used curbs (and gutter) in Florida are Type F and Type E. Type F curb is normally used on the outside edge of pavement and some raised medians and islands. Type E curb is normally used only for raised medians and islands. Details on curb types can be found in the ***Design Standards, Index 300***.

3.10 BORDER AREA

The border area provides space for a buffer between vehicles and pedestrians, sidewalks with ADA provisions, traffic control devices, fire hydrants, storm drainage features, bus and transit features, permitted public utilities and space for aesthetic features such as sod and other landscape items [***PPM***].

3.10.1 Minimum Border Widths

On highways with flush shoulders, the border is measured from the shoulder point to the right of way line (see ***Figure 3-9***). The minimum border widths for highways (arterials and collectors) are provided in ***Table 3-4***.

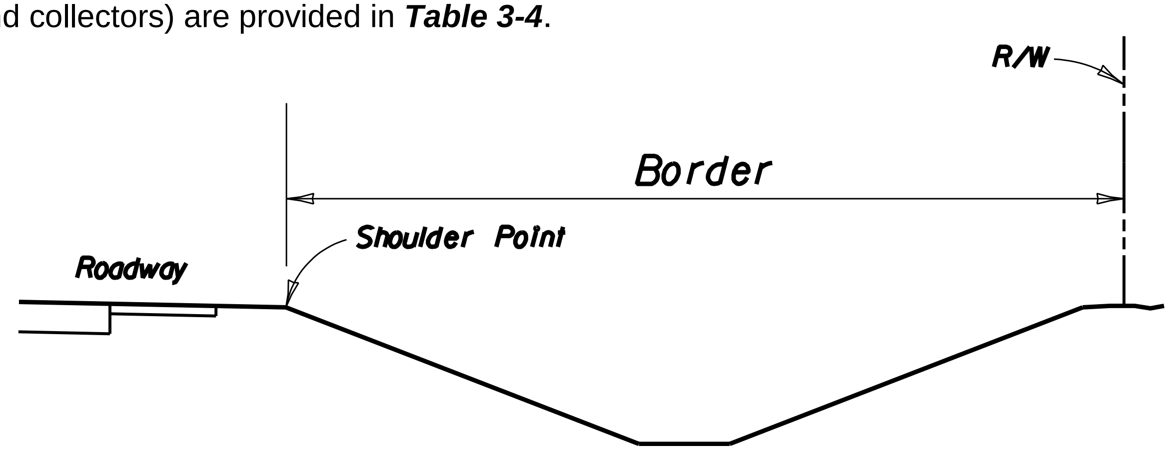


Figure 3-9 Border Width of Highways with Flush Shoulders

Table 3-4 Minimum Border Widths for Highways with Flush Shoulders [PPM]

Facility Type	Width (ft)
Arterials/Collectors (Design Speed > 45 mph)	40
Arterials/Collectors (Design Speed ≤ 45 mph)	33

On highways with curb or curb and gutter, the border is measured from the lip of the gutter (or face of curb when there is not a gutter) to the right-of-way line (see ***Figure 3-10***). The required border widths are a function of facility type, design speed and the type of lanes adjacent to the curbs, as given in the ***PPM***.

Figure 3-10 Border Width of Highways with Curb or Curb and Gutter

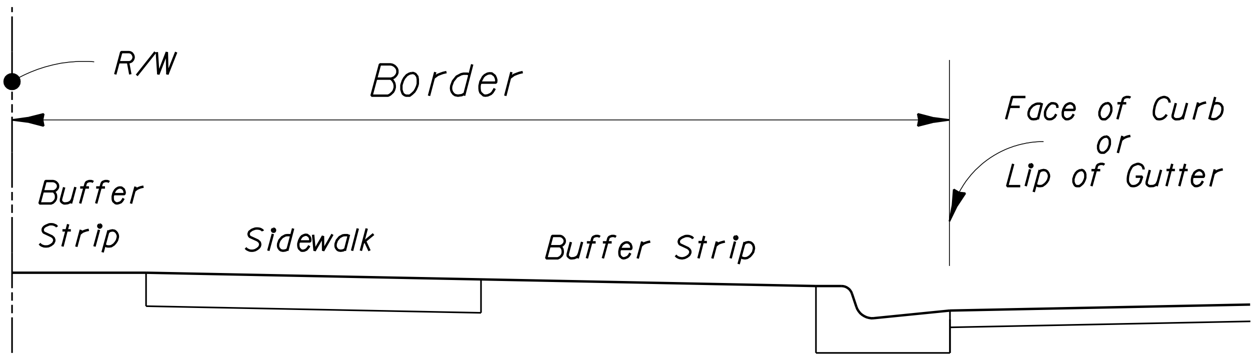


Table 3-5 Minimum Border Widths for Highways with Curbs or Curb and Gutter [*PPM*]

Facility Type	Minimum Width (ft)	
	Travel Lanes at Curb or Curb and Gutter	Bike Lanes or Other Auxiliary Lanes at Curb or Curb and Gutter
Arterials/Collectors (Design Speed = 45 mph)	14	12
Arterials/Collectors (Design Speed ≤ 40 mph)	12	10
Urban Collector Streets (Design Speed ≤ 30 mph)	10	8

On local urban streets, the border width may be a minimum of 5 feet, but desirably should be 10 feet or wider. Where the available right-of-way is limited and in areas of high right-of-way costs, as in some industrial and commercial areas, a buffer width of 2 feet may be tolerated [***AASHTO 11***].

3.10.2 Horizontal Clearance

Horizontal clearance is the lateral distance from a specified point on the roadway such as the edge of travel lane or face of curb, to a roadside feature or object. Horizontal clearance applies to all highways. Horizontal clearance requirements vary depending on design speed, whether rural or urban with curb, traffic volumes, lane type, and the feature or object.

Rural highways with flush shoulders and highways with curb or curb and gutter where right of way is not restricted have roadsides of sufficient widths to provide clear zones; therefore, horizontal clearance requirements for certain features and objects are based on maintaining a clear zone wide enough to provide recoverable terrain. See Chapter 2, Volume I of the ***PPM***. The procedure for determining required clear zone widths is described in Chapter 4, Volume I of the ***PPM***.

3.10.3 Sidewalks

Sidewalks are walkways that are parallel to the roadway and are designed for use by pedestrians. Sidewalks are generally constructed on both sides of roadways that are in or within one mile of an urban area. Sidewalks do not need to be provided when pedestrians are not expected, such as when the roadway parallels a railroad or a drainage ditch.

The minimum width of a sidewalk shall be 5 feet when separated from the curb by a buffer. The separation between the sidewalk and the back of curb shall be a minimum of 2 feet. If this separation cannot be maintained, the minimum width of the sidewalk shall be 6 feet. Sidewalks adjacent to roadways with flush shoulders shall have a minimum width of 5 feet. On existing roadways with flush shoulders, sidewalks should be placed as far from the roadway as practical. For new roadways with flush shoulders, the sidewalk should be constructed outside the clear zone.

Sidewalk grades should not exceed 5% when not adjacent to the roadway. The sidewalk cross slope should allow for adequate drainage, but shall be no more than 2% to comply with **ADA** requirements.

3.10.4 Curb Ramps

Pedestrian facilities must be designed in accordance with **ADA** to accommodate the visually and physically challenged citizens whose mobility is dependent on wheelchairs and other devices. In areas with sidewalks, curb ramps with tactile surfaces shall be incorporated at all locations where a marked crosswalk adjoins the sidewalk. ***Design Standards, Index 304*** sets forth the requirements [***PPM***].

To assist pedestrians who are visually or mobility impaired, curb ramps should be parallel to the crossing. By providing ramps parallel to the crossing, the pedestrian is directed into the crossing. At intersections where more than one road is crossed, each crossing should have a separate curb ramp. Under no circumstances should a curb ramp be installed allowing a pedestrian to enter a crossing without providing a curb cut (or at grade sidewalk if no curb is present) on the opposite side of the crossing [***PPM***].

3.11 Channelizing Islands

Channelizing Islands provide for the separation of conflicting traffic movements into defined paths of travel to facilitate the safe and orderly movement of vehicles, pedestrians and bicycles. Islands serve three primary functions: (1) channelization—to control and direct traffic movement, usually turning; (2) division—to divide opposing or same direction traffic streams, usually through movements and (3) refuge—to provide refuge for pedestrians. Most islands combine two or all of these functions. Islands may range from an area delineated by a raised curb to a pavement area marked out by paint or thermoplastic markings. An example island is shown in ***Figure 3-11***.

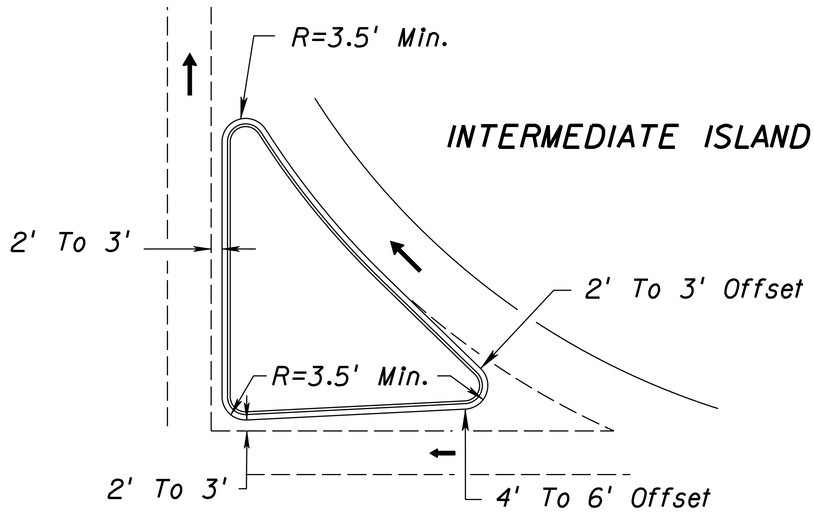


Figure 3-11 Typical Triangular Curbed Island

Proper channelization increases capacity, improves safety, and provides positive guidance to motorists. Improper channelization has the opposite effect and may be worse than none at all [***AASHTO 11***]. The general guidelines for channelization design include:

1. Conflicts should be separated so that drivers and pedestrians are confronted with only one conflict and make only one decision at a time.
2. The proper traffic channels should seem obvious, easy to follow, and of unquestionable continuity.
3. Unnatural paths that involve turns greater than 90° or sudden and sharp reverse curves should be avoided.
4. Areas of conflict should be reduced as much as practical while merging and weaving areas should be maximized.
5. The angle of intersection between merging streams of traffic should be appropriate to provide adequate sight distance.
6. The number of islands should be held to a practical minimum to avoid confusion.
7. Islands should meet minimum size requirements.
8. Channelization must be visible.
9. All movements must be accounted for.
10. The major traffic flows should be favored.

11. Islands should be designed for the design speed.
12. The approach end treatment and delineation should be carefully designed to be consistent with the speed characteristics of the roadway design.

3.11.1 Island Sizes

Islands should be sufficiently large to command attention. The smallest curbed island should have an area of approximately 50 feet^2 for urban and 75 feet^2 for rural intersections. However, 100 feet^2 is preferable for both. Accordingly, triangular islands should not be less than about 12 feet and preferably 15 feet, on a side after rounding of corners. Large curbed islands with side dimensions of at least 100 feet should not be used on high-speed facilities. [***AASHTO 11***].

Divisional islands should be not less than 4 feet wide and 20 to 25 feet long. In general, introducing curbed divisional islands at isolated intersections on high-speed highways is undesirable unless special attention is directed towards providing high visibility for the islands. Curbed divisional islands at these intersections should be 100 feet or more in length. When situated in the vicinity of a high point in the roadway profile or at or near the beginning of a horizontal curve, the approach end of the curbed islands should be extended to be clearly visible to approaching drivers [***AASHTO 11***].

3.11.2 Island Delineation

Islands should be delineated or outlined by a variety of treatments, depending on their size, location and function. Island delineation can be divided into three types: (1) raised-curb islands, (2) islands delineated by pavement markings or reflectorized markers placed on paved areas and, (3) islands formed by the pavement edges, possibly supplemented by flexible delineators or other flexible guideposts, or a mounded-earth treatment beyond and adjacent to the pavement edges.

Delineation of small islands is effected primarily by curbs. Large curbed islands may be sufficiently delineated by color and texture contrast of vegetative cover, mounded earth, shrubs, guideposts, tubular markers, signs or any combination of these. On rural highways with design speeds of 50 mph or greater, curbed islands should not be used. Standard markings for islands are provided in the ***Design Standards, Index 17346***.

3.11.3 Approach-End Treatment

The outline of a curbed island is determined by the edge of through-traffic lanes or turning roadways, with lateral clearance, if any, to the face of the curbed island. The end-points are rounded or beveled for visibility and construction simplicity. The approach and departure noses are rounded with radii of at least 3.5 feet. ***Figure 3-12*** illustrates two common island configurations, one with paved shoulders and one with a parallel offset.

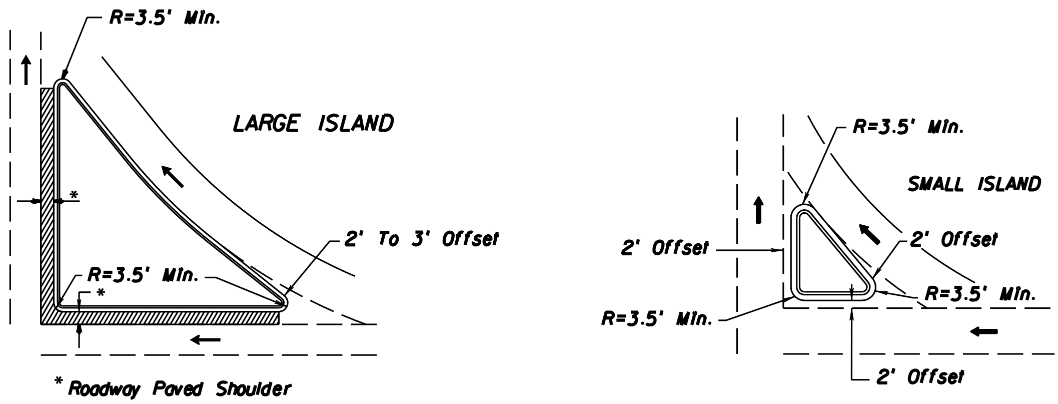


Figure 3-12 Curbed Island Configurations

Approach ends of the island should be offset from the edges of the traveled way in order to funnel drivers smoothly into the desired path. Failure to offset approach ends can make an island appear more restrictive than it actually is and can have a psychological effect on drivers, causing them to make erratic movements as they approach the intersection. The amount that a curbed island is offset from the through-traffic lane is influenced by the type of edge treatment and other factors such as island contrast, length of taper or auxiliary pavement preceding the curbed island. [***AASHTO 11***]. However, where there is a bike lane adjacent to an island curb, no offset is needed.

Where there are no curbs on the approach traveled way, the minimum offset of the edge of the curbed island to the through lane should be 1.5 to 3.5 feet. Where the approach roadway has a Type E curb, a similar curb on the curbed island could be located at the edge of the through lane where there is sufficient length of curbed island to affect a gradual taper from the nose offset. Type F curbs should be offset from the through traveled way edge, regardless of the size of the curbed island, to avoid a sense of lateral restriction to drivers. For intermediate and large-size islands that are uncurbed, offsets are desirable but not essential. However, any fixed objects within the island areas should be offset an appropriate distance from the through lanes.

3.11.4 Design of Median Islands/Traffic Separators

Design of median islands generally reflects site-specific geometrics such as angle of intersection and cross section. In addition to meeting the requirements set forth above, as appropriate, design of median islands should meet the following guidelines [***NCHRP 279***]:

1. Approach noses should be offset 2 to 6 feet from the through (approach) lanes to minimize accidental impacts. Pavement markings in advance of the nose can be used to transition from the centerline to the edge of island.
2. The shape of the island should be based on design turning paths and the island function. Curvilinear tapers comprised of parabolic or circular curves generally suffice.
3. The length of the island should be related to the approach speed. An estimate is to use the length based on 3-second driving time to the intersection.

4. The width of the island should adequately serve its intended functions, as given previously in ***Table 3-3***.
5. Median islands should begin on tangent alignment and on upgrades or well past crest vertical curves. In some cases it is appropriate to extend a median island to avoid its introduction on a horizontal curve or within an area of limiting sight distance.

Design Standards, Index 302 provides detailed dimensional design for traffic separators.

3.11.5 Splitter Islands

Splitter Islands refer to divisional islands on the approaches to roundabouts. Inasmuch as roundabouts are designed to reduce speeds through lane deflection, splitter islands channelize vehicles into a curvilinear path through the circulatory roadway around the central island. Design of splitter islands must be accomplished during the overall layout of the roundabout, based on the selected design vehicle and its maneuverability, without compromising the speed reduction effect to passenger vehicles. In general, approach treatments to splitter islands should be accomplished in the same manner as median islands and traffic separators.

3.11.6 Refuge Islands

A refuge island for pedestrians is one at or near a crosswalk or shared use paths that aids and protects pedestrians and bicyclists who cross the roadway. Raised curb corner islands and center channelizing or divisional islands can be used as refuge areas. Refuge islands for pedestrians and bicyclists crossing a wide street for loading or unloading transit riders or for wheelchair ramps are used primarily in urban areas. Splitter islands in roundabouts are intended to provide pedestrian refuge and should be no less than six feet in width.

The general principles for channelization island design also apply directly to providing refuge islands. The location and width of crosswalks, the location and size of transit loading zones and the provisions for wheelchair ramps influence the size and location of refuge islands. Refuge islands should be a minimum of 6 feet wide when they will be used for bicyclists. Pedestrians and bicyclists should have a clear path through the island and should not be obstructed by poles, sign posts, utility boxes, etc.

3.12 AUXILIARY LANES

Auxiliary lanes are used at intersections preceding median openings for left-turning movements, preceding and following right-turning movements, and in some cases for through movements. The primary function of auxiliary lanes at intersections is to accommodate speed changes and maneuvering of turning traffic. They may also be added to increase capacity through an intersection. The minimum widths for auxiliary lanes are given in ***Table 2.1.1***, .***Volume I***, ***Chapter 2*** of the ***PPM***.

The length of the auxiliary lanes consists of three components: (1) deceleration length, (2) storage or queue length and (3) entering taper as shown in ***Figure 3-13***. The total length of the auxiliary lane is the sum of the lengths of these three components. It is common practice to accept a moderate amount of deceleration within the through lanes and to consider the taper as part of the deceleration length. The length criteria for each of the auxiliary lane components are given below.

3.12.1 Deceleration Length

The required total deceleration length is that needed for a safe and comfortable stop from the design speed of the highway. Minimum deceleration lengths (including taper) for auxiliary lanes must comply with ***Table 3-6*** from the ***Design Standards, Index 301***.

3.12.2 Storage (Queue) Length

The auxiliary lane should be sufficiently long to store the number of vehicles likely to accumulate during a critical period. The storage length should be sufficient to avoid the possibilities of turning vehicles stopping in the through lanes or the entrance to the auxiliary lane being blocked by vehicles queuing in the through lanes.

At unsignalized intersections the storage length, exclusive of taper, may be based on the number of turning vehicles likely to arrive in an average two-minute period within the peak hour. Space for at least two passenger cars should be provided, with over 10% truck traffic provisions should be made for at least one car and one truck.

At signalized intersections the required storage length depends on the signal cycle length, the signal phasing arrangement and the rate of arrivals and departures of turning vehicles. The storage length is a function of the probability of occurrence of events and should be based on 1.5 to 2 times the average number of vehicles that would store per cycle, which is predicted on the design volume. Chapter 4 provides additional details on accommodating queues at signalized intersections.

Where dual turning lanes are used, the storage length is reduced to approximately one-half of that required for single-lane operation [***AASHTO 11***].

3.12.3 Approach End Taper

The constructed length of approach end tapers shall be 50 feet for a single turn lane and 100 feet for a double turn lane, as shown in ***Figure 3-13***. These constructed taper lengths apply to all design speeds. These relatively short taper lengths are intended to provide approaching road users with positive identification of an added auxiliary lane and results in a longer full width auxiliary lane than use of longer taper lengths based on the path that road users actually follow. The clearance distances L_1 and L_3 account for the full transition lengths a road user will use to enter the auxiliary lane for various speed conditions assumed for design.

Table 3-6 Minimum Deceleration Lengths

Turn Lanes -- Curbed and Uncurbed Medians								
			Urban Conditions			Rural Conditions		
Design Speed (mph)	Entry Speed (mph)	Clearance Distance L_1 (feet)	Brake To Stop Distance L_2 (feet)	Total Decel. Distance L (feet)	Clearance Distance L_3 (feet)	Brake To Stop Distance L_2 (feet)	Total Decel. Distance L (feet)	Clearance Distance L_3 (feet)
35	25	70	75	145	110	----	----	----
40	30	80	75	155	120	----	----	----
45	35	85	100	185	135	----	----	----
50	40/44	105	135	240	160	185	290	160
55	48	125	----	----	----	225	350	195
60	52	145	----	----	----	260	405	230
65	55	170	----	----	----	290	460	270

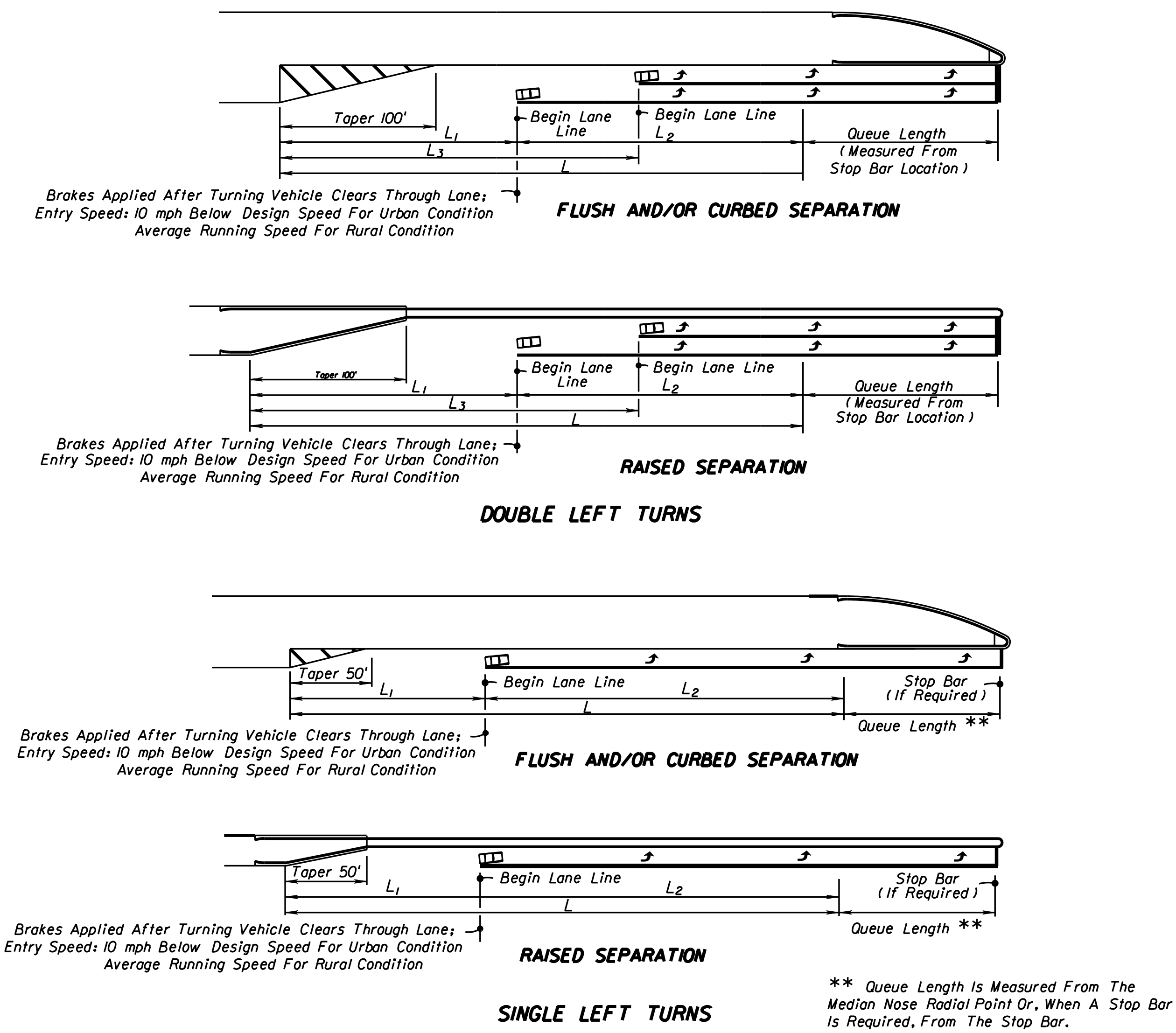


Figure 3-13 Turn Lanes [*Design Standards, Index 301*]

3.13 TURNING ROADWAYS

In most cases, turning roadways are designed for use by right turning traffic. There are three basic types of right turning roadways at intersections: (1) a minimum edge of traveled way design, (2) a design with a corner triangular island, (3) a free flow design using a simple radius or compound radii. The turning radii and the pavement cross slopes for free-flow right turns are functions of design speed and type of vehicles.

3.13.1 Minimum Edge of Traveled Way Design

When turning vehicles are to be accommodated within minimum space, corner radii should be based on the minimum turning paths of the selected design vehicles. ***Table 3-7*** and ***Table 3-8*** summarize the minimum edge of traveled way design for various design vehicles. These minimum designs provide for the minimum turning paths attainable at speeds equal to or less than 10 mph. ***Figure 3-14*** demonstrates the angle of reference for use in these tables.

In urban areas, corner radii should satisfy the needs of the road users using them, the amount of right of way available, the angle of turn between intersection legs, the number of pedestrians using the crosswalk, the width and number of lanes on the intersecting street and the speeds on each street. Guidelines for corner radii in urban areas are as follows:

1. Radii of 15 to 25 feet are adequate for passenger vehicles. These radii are suitable for minor cross streets where there is little occasion for trucks to turn and at major intersections where there are parking lanes.
2. Radii of 25 feet or more should be provided at minor cross streets, on new construction and on reconstruction projects.
3. Radii of 30 feet or more should be provided at minor cross streets where practical so that an occasional truck can turn without too much encroachment.
4. Radii of 40 feet or more or preferably three-centered curves or simple curves with tapers to fit the paths of large truck combinations, should be provided where such combinations or buses turn frequently. Where speed reductions would cause problems, longer radii should be considered.
5. Curb radii should be coordinated with crosswalk distances or special designs should be used to make crosswalks efficient for all pedestrians. Where larger radii are used, an intermediate refuge or median island is desirable or crosswalks may need to be offset so that crosswalk distances are not objectionable.

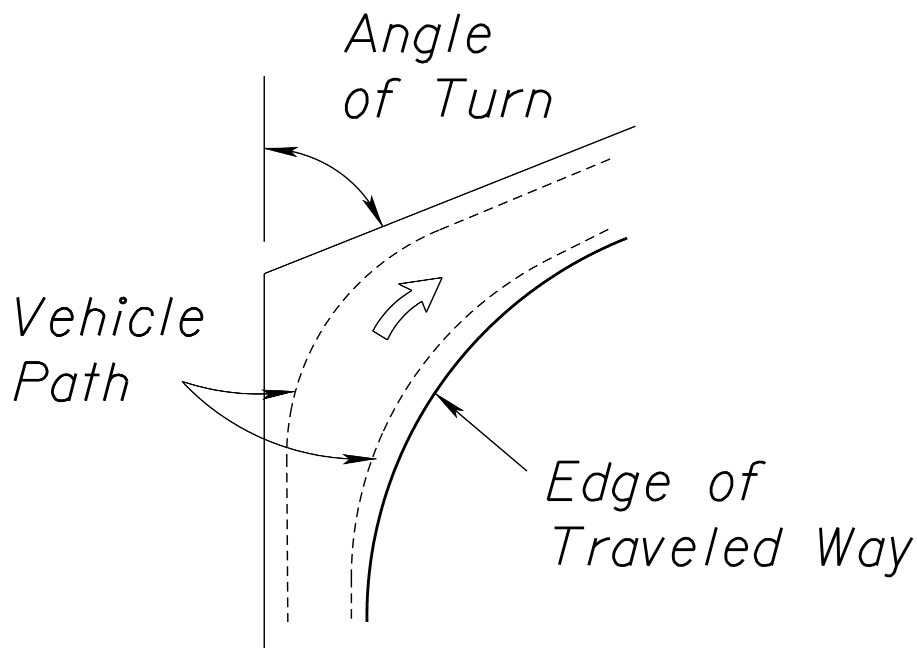


Figure 3-14 Reference Turn Angle for Turning Roadway Designs

Table 3-7 Edge-of-Traveled-Way Designs for Turns at Intersections

Angle of turn (degrees)	Design vehicle	Simple curve radius (feet)	Simple curve radius with taper		
			Radius (feet)	Offset (feet)	Taper H:V
30	P	60	----	----	----
	SU-30	100	----	----	----
	SU-40	140	----	----	----
	WB-40	150	----	----	----
	WB-62	360	220	3.0	15:1
	WB-62FL	380	220	3.0	15:1
	WB-67	380	220	3.0	15:1
	WB-92D	365	190	3.0	15:1
	WB-100T	260	125	3.0	15:1
	WB-109D	475	260	3.5	20:1
45	P	50	----	----	----
	SU-30	75	----	----	----
	SU-40	115	----	----	----
	WB-40	120	----	----	----
	WB-62	230	145	4.0	15:1
	WB-62FL	250	145	4.5	15:1
	WB-67	250	145	4.5	15:1
	WB-92D	270	145	4.0	15:1
	WB-100T	200	115	2.5	15:1
	WB-109D	----	200	4.5	20:1
60	P	40	----	----	----
	SU-30	60	----	----	----
	SU-40	100	----	----	----
	WB-40	90	----	----	----
	WB-62	170	140	4.0	15:1
	WB-62FL	200	140	4.5	15:1
	WB-67	200	140	4.5	15:1
	WB-92B	230	120	5.0	15:1
	WB-100T	150	95	2.5	15:1
	WB-109D	----	180	4.5	20:1
75	P	35	25	2.0	10:1
	SU-30	55	45	2.0	10:1
	SU-40	90	60	2.0	10:1
	WB-40	----	60	2.0	15:1
	WB-62	----	145	4.0	20:1
	WB-62FL		145	4.0	20:1
	WB-67	----	145	4.5	20:1
	WB-92D	----	110	5.0	15:1
	WB-100T	----	85	3.0	15:1
	WB-109D	----	140	5.5	20:1

Table 3-7 Edge-of-Traveled-Way Designs for Turns at Intersections (Continued)

Angle of turn (degrees)	Design vehicle	Simple curve radius (feet)	Simple curve radius with taper: Radius (feet)	Offset (feet)	Taper H:V
90	SU-30	50	40	2.0	10:1
	SU-40	80	45	4.0	10:1
	WB-40	----	45	4.0	10:1
	WB-62	----	120	4.5	30:1
	WB-62FL		125	4.5	30:1
	WB-67	----	125	4.5	30:1
	WB-92D	----	95	6.0	10:1
	WB-100T	----	85	2.5	15:1
	WB-109D	----	115	2.9	15:1
105	P	----	20	2.5	8:1
	SU-30	----	35	3.0	10:1
	SU-40	----	45	4.0	10:1
	WB-40	----	40	4.0	10:1
	WB-62	----	115	3.0	15:1
	WB-62FL		115	3.0	15:1
	WB-67	----	115	3.0	15:1
	WB-92D	----	80	8.0	10:1
	WB-100T	----	75	3.0	15:1
	WB-109D	----	90	9.2	20:1
120	P	----	20	2.0	10:1
	SU-30	----	30	3.0	10:1
	SU-40	----	35	6.0	8:1
	WB-40	----	35	5.0	8:1
	WB-62	----	100	5.0	15:1
	WB62FL		105	5.2	15:1
	WB-67	----	105	5.2	15:1
	WB-92D	----	80	7.0	10:1
	WB-100T	----	65	3.5	15:1
	WB-109D	----	85	9.2	20:1
135	P	----	20	1.5	10:1
	SU-30	----	30	4.0	10:1
	SU-40	----	40	4.0	8:1
	WB-40	----	30	8.0	15:1
	WB-62	----	80	5.0	20:1
	WB-62FL		85	5.2	20:1
	WB-67	----	85	5.2	20:1
	WB-92D	----	75	7.3	10:1
	WB-100T	----	65	5.5	15:1
	WB-109D	----	85	8.5	20:1

Table 3-7 Edge-of-Traveled-Way Designs for Turns at Intersections (Continued)

Angle of turn (degrees)	Design vehicle	Simple curve radius (feet)	Simple curve radius with taper		
			Radius (feet)	Offset (feet)	Taper H:V
150	P	----	18	2.0	10:1
	SU-30	----	30	4.0	8:1
	SU-40	----	35	7.0	8:1
	WB-40	----	30	6.0	8:1
	WB-62	----	60	10.0	10:1
	WB-62FL		65	10.2	10:1
	WB-67	----	65	10.2	10:1
	WB-92D	----	65	11.0	10:1
	WB-100T	----	65	7.3	10:1
	WB-109D	----	65	15.1	10:1
180	P	----	15	0.5	20:1
	SU-30	----	30	1.5	10:1
	SU-40				
	WB-40	----	20	9.5	5:1
	WB-62	----	55	10.0	15:1
	WB-62FL	----	55	13.8	10:1
	WB-67	----	55	13.8	10:1
	WB92D	----	55	16.8	10:1
	WB-100T	----	55	10.2	10:1
	WB-109D	----	55	20.0	10:1

Table 3-8 Edge-of-Traveled-Way Designs for Turns at Intersections

Angle of turn (degrees)	Design vehicle	3-Centered compound			
		Curve radii (feet)	Symmetric offset (feet)	Curve radii (feet)	Asymmetric (feet)
30	P	----	----	----	----
	SU-30	----	----	----	----
	SU-40	----	----	----	----
	WB-40	----	----	----	----
	WB-62	----	----	----	----
	WB-62FL	460-175-460	4.0	300-175-550	2.0-4.5
	WB-67	460-175-460	4.0	300-175-550	2.0-4.5
	WB-92D	550-155-550	4.0	200-150-500	2.0-6.0
	WB-100T	220-80-220	4.5	200-80-300	2.5-5.0
	WB-109D	550-250-550	5.0	250-200-650	1.5-7.0
45	P	----	----	----	----
	SU-30	----	----	----	----
	SU-40	----	----	----	----
	WB-40	----	----	----	----
	WB-62	460-240-460	2.0	120-140-500	3.0-8.5
	WB-62FL	460-175-460	4.0	250-125-600	1.0-6.0
	WB-67	460-175-460	4.0	250-125-600	1.0-6.0
	WB-92D	525-155-525	5.0	200-140-500	1.5-6.0
	WB-100T	250-80-250	4.5	200-80-300	2.5-5.5
	WB-109D	550-200-550	5.0	200-170-650	1.5-7.0
60	P	----	----	----	----
	SU-30	----	----	----	----
	SU-40	----	----	----	----
	WB-40	----	----	----	----
	WB-62	400-100-400	15.0	110-100-220	10.0-12.5
	WB-62FL	400-100-400	8.0	250-125-600	1.0-6.0
	WB-67	400-100-400	8.0	250-125-600	1.0-6.0
	WB-92D	480-110-480	6.0	150-110-500	3.0-9.0
	WB-100T	250-80-250	4.5	200-80-300	2.0-5.5
	WB-109D	650-150-650	5.5	200-140-600	1.5-8.0
75	P	100-25-100	2.0	----	----
	SU-30	120-45-120	2.0	----	----
	SU-40	200-35-200	5.0	60-45-200	1.0-4.5
	WB-40	120-45-120	5.0	120-45-195	2.0-6.5
	WB-62	440-75-440	15.0	140-100-540	5.0-12.0
	WB-62FL	420-75-420	10.0	200-80-600	1.0-10.0
	WB-67	420-75-420	10.0	200-80-600	1.0-10.0
	WB-92D	500-95-500	7.0	150-100-500	1.0-8.0
	WB-100T	250-80-250	4.5	100-80-300	1.5-5.0
	WB-109D	700-125-700	6.5	150-110-550	1.5-11.5
90	P	100-20-100	2.5	----	----
	SU-30	120-40-120	2.0	----	----
	SU-40	200-30-200	7.0	60-45-200	1.0-4.5
	WB-40	120-40-120	5.0	120-40-200	2.0-6.5

Angle of turn (degrees)	Design vehicle	3-Centered compound			
		Curve radii (feet)	Symmetric offset (feet)	Curve radii (feet)	Asymmetric (feet)
	WB-62	400-70-400	10.0	160-70-360	6.0-10.0
	WB-62FL	440-65-440	10.0	200-70-600	1.0-11.0
	WB-67	440-65-440	10.0	200-70-600	1.0-11.0
	WB-92D	470-75-470	10.0	150-90-500	1.5-8.5
	WB-100T	250-70-250	4.5	200-70-300	1.0-5.0
	WB-109D	700-110-700	6.5	100-95-550	2.0-11.5
105	P	100-20-100	2.5	----	----
	SU-30	100-35-100	3.0	----	----
	SU-40	200-35-100	6.0	60-40-190	1.5-6.0
	WB-40	100-35-100	5.0	100-55-200	2.0-8.0
	WB-62	520-50-520	15.0	360-75-600	4.0-10.5
	WB-62FL	500-50-500	13.0	200-65-600	1.0-11.0
	WB-67	500-50-500	13.0	200-65-600	1.0-11.0
	WB-92D	500-80-500	8.0	150-80-500	2.0-10.0
	WB-100T	250-60-250	5.0	100-60-300	1.5-6.0
	WB-109D	700-95-700	8.0	150-80-500	3.0-15.0
120	P	100-20-100	2.0	----	----
	SU-30	100-30-100	3.0	----	----
	SU-40	200-35-200	6.0	60-40-190	1.5-5.0
	WB-40	120-30-120	6.0	100-30-180	2.0-9.0
	WB-62	520-70-520	10.0	80-55-520	24.0-17.0
	WB-62FL	550-45-550	15.0	200-60-600	2.0-12.5
	WB-67	550-45-550	15.0	200-60-600	2.0-12.5
	WB-92D	500-70-500	10.0	150-70-450	3.0-10.5
	WB-100T	250-60-250	5.0	100-60-300	1.5-6.0
	WB-109D	700-85-700	9.0	150-70-500	7.0-17.4
135	P	100-20-100	1.5	----	----
	SU-30	100-30-100	4.0	----	----
	SU-40	200-40-200	4.0	60-40-180	1.5-5.0
	WB-40	120-30-120	6.5	100-25-180	3.0-13.0
	WB-62	600-60-600	12.0	100-60-640	14.0-7.0
	WB-62FL	550-45-550	16.0	200-60-600	2.0-12.5
	WB-67	550-45-550	16.0	200-60-600	2.0-12.5
	WB-92D	450-70-450	9.0	150-65-450	7.0-13.5
	WB-100T	250-60-250	5.5	100-60-300	2.5-7.0
	WB-109D	700-70-700	12.5	150-65-500	14.0-18.4
150	P	75-20-75	2.0	----	----
	SU-30	100-30-100	4.0	----	----
	SU-40	200-35-200	6.5	60-40-200	1.0-4.5
	WB-40	100-30-100	6.0	90-25-160	1.0-12.0
	WB-62	480-55-480	15.0	140-60-560	8.0-10.0
	WB-62FL	550-45-550	19.0	200-55-600	7.0-16.4
	WB-67	550-45-550	19.0	200-55-600	7.0-16.4
	WB-92D	350-60-350	15.0	120-65-450	6.0-13.0
	WB-100T	250-60-250	7.0	100-60-300	5.0-8.0
	WB-109D	700-65-700	15.0	200-65-500	9.0-18.4

Angle of turn (degrees)	Design vehicle	3-Centered compound			
		Curve radii (feet)	**Symmetric offset (feet)**	**Curve radii (feet)**	**Asymmetric (feet)**
180	P	50-15-50	0.5	----	----
	SU-30	100-30-100	1.5	----	----
	SU-40	150-35-150	6.2	50-35-130	5.5-7.0
	WB-40	100-20-100	9.5	85-20-150	6.0-13.0
	WB-62	800-45-800	20.0	100-55-900	15.0-15.0
	WB-62FL	600-45-600	20.5	100-55-400	6.0-15.0
	WB-67	600-45-600	20.5	100-55-400	6.0-15.0
	WB-92D	400-55-400	16.8	120-60-400	9.0-14.5
	WB-100T	250-55-250	9.5	100-55-300	8.5-10.5
	WB-109D	700-55-700	20.0	200-60-500	10.0-21.0

Table 3-9 summarizes the operational characteristics of various corner radii for the range of design vehicles.

Table 3-9 Operational Characteristics of Corner Radii [*NCHRP 279*]

Corner Radius (ft)	Operational Characteristics
< 5	Not appropriate for even P design vehicles, except for approaches where right turns are prohibited because of one-way streets
10	Crawl speed turn for P vehicles
20 - 30	Low speed turn for P vehicles, crawl speed turn for SU vehicle with minor lane encroachment
40	Moderate speed turn for P vehicle, low speed turn for SU vehicle, crawl speed turn for WB-40 vehicle with minor encroachment
50	Moderate speed turns for all vehicles up to WB-40

3.13.2 Turning Roadways with Corner Islands

Where the inner edges of the traveled way for right turns are designed to accommodate semi-trailer combinations or where the design permits passenger vehicles to turn at speeds greater than 10 mph, the pavement area within the intersection may become excessively large and does not provide for proper control of traffic. To avoid this condition, a corner island can be provided to form a separate turning roadway. Section 3.11 provides information on the design of islands.

3.13.3 Turning Roadways - Free Flow Design

An important part of the design on some intersections is the design of a free flow alignment for turns. Ease and smoothness of operation can result when the free flowing turning roadway is designed with compound curves preceded by a deceleration lane. Turning radii and pavement cross slope for free flow right turns at speeds greater than 10 mph are a function of the design speed and type of vehicles to be accommodated. In general, the design speed of the turning roadway should be equal to, or within 10 to 20 mph less than the through roadway design speed.

Within an intersection, motorists anticipate sharp curves and will accept operation with higher side friction than they will on open highway curves of the same radii, when their speed is not affected by other vehicles. It is desirable to provide as much superelevation as practical on intersection curves, particularly where the intersection curve is sharp and on a downgrade. However, the short curvature and short lengths of turning roadways often prevents the development of a desirable amount of superelevation. ***Table 3-10*** gives the minimum superelevation rates in relation to design speed. The wide variation in likely speeds on intersection curves precludes need for precision, so only the minimum superelevation rate is given for each design speed and intersection curve radius.

Table 3-10 Superelevation Rates for Curves at Intersections

	Design Speed (MPH)							
	10	**15**	**20**	**25**	**30**	**35**	**40**	**45**
Minimum Superelevation Rate	0.00*	0.00*	0.02	0.04	0.06	0.08	0.09	0.10
Minimum Radius (FEET)	25	50	90	150	230	310	430	540

* The rate of 0.02 is considered the practical minimum for effective drainage across the surface.

Note: It is preferable to use superelevation rates greater than these minimum values.

3.13.4 Minimum Widths

The widths of turning roadways at intersections are governed by the volumes of turning traffic and the types of vehicles to be accommodated and may be designed for one-way or two-way operation, depending on the geometric pattern of intersection. The ***AASTHO Green Book*** classifies turning roadways into the following types of operations:

Case I: One-lane, one-way operation with no provision for passing a stalled vehicle.
Case II: Same as case I but with provision for passing a stalled vehicle.
Case III: Two-lane operation, either one-way or two-way operation.

The design widths for turning roadways for different design vehicles for each of the above cases are given in ***Table 3-11***.

Widths under Case I are typically used for minor turning movements and for moderate turning volumes where the connecting roadway is relatively short.

Widths under Case II are determined to allow operation at low speed and with sufficient clearance so that other vehicles can pass a stalled vehicle. These widths are applicable to all turning movements of moderate to heavy traffic volumes that do not exceed the capacity of a single-lane connection. In the event of a breakdown, traffic flow can be maintained at somewhat reduced speed.

Widths under Case III are applicable where operation is two-way or where operation is one-way but two lanes are needed to handle the traffic volume.

Table 3-11 Design Widths for Turning Roadways [*AASHTO 2011*]

Radius on Inner Edge of Pavement (feet)	Pavement Width (feet)								
	Case I One-Lane, One-Way Operation – No Provision for Passing a Stalled Vehicle			Case II One-Lane, One-Way Operation – With Provision for Passing a Stalled Vehicle			Case III Two-Lane Operation – Either One-Way or Two-Way		
	Design Traffic and Condition								
	A	**B**	**C**	**A**	**B**	**C**	**A**	**B**	**C**
50	18	18	23	20	26	30	31	36	45
75	16	17	20	19	23	27	29	33	38
100	15	16	18	18	22	25	28	31	35
150	14	15	17	18	21	23	26	29	32
200	13	15	16	17	20	22	26	28	30
300	13	15	15	17	20	22	25	28	29
400	13	15	15	17	19	21	25	27	28
500	12	15	15	17	19	21	25	27	28
Tangent	12	14	14	17	18	20	24	26	26

Note: Traffic Condition A = predominately P vehicles, but some consideration for SU trucks.
Traffic Condition B = sufficient SU-30 vehicles to govern design, but some consideration for semitrailer combination trucks.
Traffic Condition C = sufficient bus and combination-trucks to govern design.

For guidance on width modifications for various edge treatments, see Table 3-29, ***AASHTO*** 2011.

3.13.5 Cross Slopes

The maximum algebraic difference in cross slope at turning roadway terminals (shown in ***Figure 3-15*** should satisfy the values given in ***Table 3-12***.

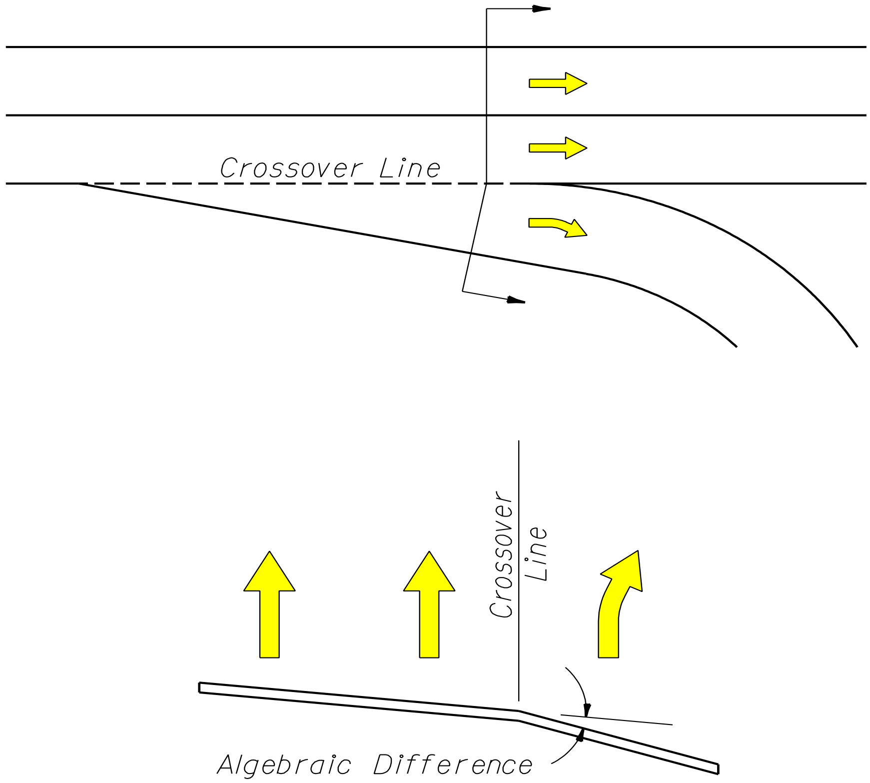


Figure 3-15 Maximum Algebraic Difference at Turning Roadway Terminals

Table 3-12 Maximum Algebraic Difference in Cross Slope at Turning Roadway Terminals

Design Speed of Exit or Entrance Curve (mph)	Maximum Algebraic Difference in Cross Slope at Crossover Line (%)
Less than 35	6.0
35 and over	5.0

3.13.6 Control Radii for Minimum Turning Path

Control radius refers to a radius that must be considered in establishing the location of median and/or traffic separator ends on divided highways and the stop bar on undivided highways. For most intersections this radius is provided for left turning movements. Design guidance on minimum edge of traveled way design for various design vehicles is provided in Section 3.13.1. This guidance is applicable to left turns as well as right turns at intersections and should be used where there is a physical edge of traveled way for left turns, as in a channelized intersection. However, typical intersections do not have a continuous physical edge of traveled way delineating the left turn path. Instead, the road user has guides at the beginning and end of the left turn operation: 1) the centerline of an undivided crossroad or the median edge of a divided crossroad and 2) the median end. For the central part of the turn the road user has the open central intersection area in which to maneuver. Under these circumstances, the use of compound curves is not necessary and the use of simple curves is satisfactory. ***Table 3-13*** provides control radii for minimum speed turns (10 to 15 mph) that can be used for establishing the location of the median ends.

Table 3-13 Control Radii for Minimum Speed Turns

Design Vehicles Accommodated	Control Radius (feet)			
	50 (40 min)	60 (50 min)	75	130
Predominant	P	SU-30	SU-40, WB-40, WB-62	WB-62FL
Occasional	SU-30	SU-40, WB-40	WB-62FL	WB-67

3.13.7 Double or Triple Left and Double Right Turning Lanes

When double or triple left turn or double right turn lanes are provided, special consideration must be given to providing turning radii to accommodate two or three vehicles turning abreast. The radius of curvature in combination with the track width of the selected design vehicle(s) will establish the required width within the turn. Lane lines (or guide lines) and width requirements should be determined by plotting the swept paths of the selected design vehicles. For most intersections on the SHS, design of double or triple lane turns should consider as a minimum one SU-40 vehicle(s) and one P vehicle turning simultaneously as shown in ***Figure 3-16***. Triple left turns should be designed to accommodate one WB-62FL, one SU-40, and one P vehicle turning simultaneously.

For left turns, the selection of an appropriate control radius for the inside turning lane should be based on the guidance in Section 3.13.6. The inside edge of the outer lane should be based on providing a minimum 4 feet separation between the swept path of the selected design vehicles in each lane traveling in the same direction. Except for turns with large radii, the inside edge of the outer lane will not be concentric with the selected control radius. An appropriate radius for the inside edge of the outer turn lane should be determined by analysis of the plotted swept path of the design vehicles. The separation between vehicles traveling in opposing directions simultaneously should desirably be 8 feet or more. Lesser separation may be acceptable where turning paths are highly visible and speeds are low. If adequate separation cannot be achieved, separate left turn phases for each direction will be necessary.

A similar approach should be used for double right turns, with the radius of the inside right turn lane based on the guidance in Sections 3.13.1, 3.13.2 and 3.13.3. For right turn lanes separated by traffic islands, see also Section 3.13.4 for minimum width requirements.

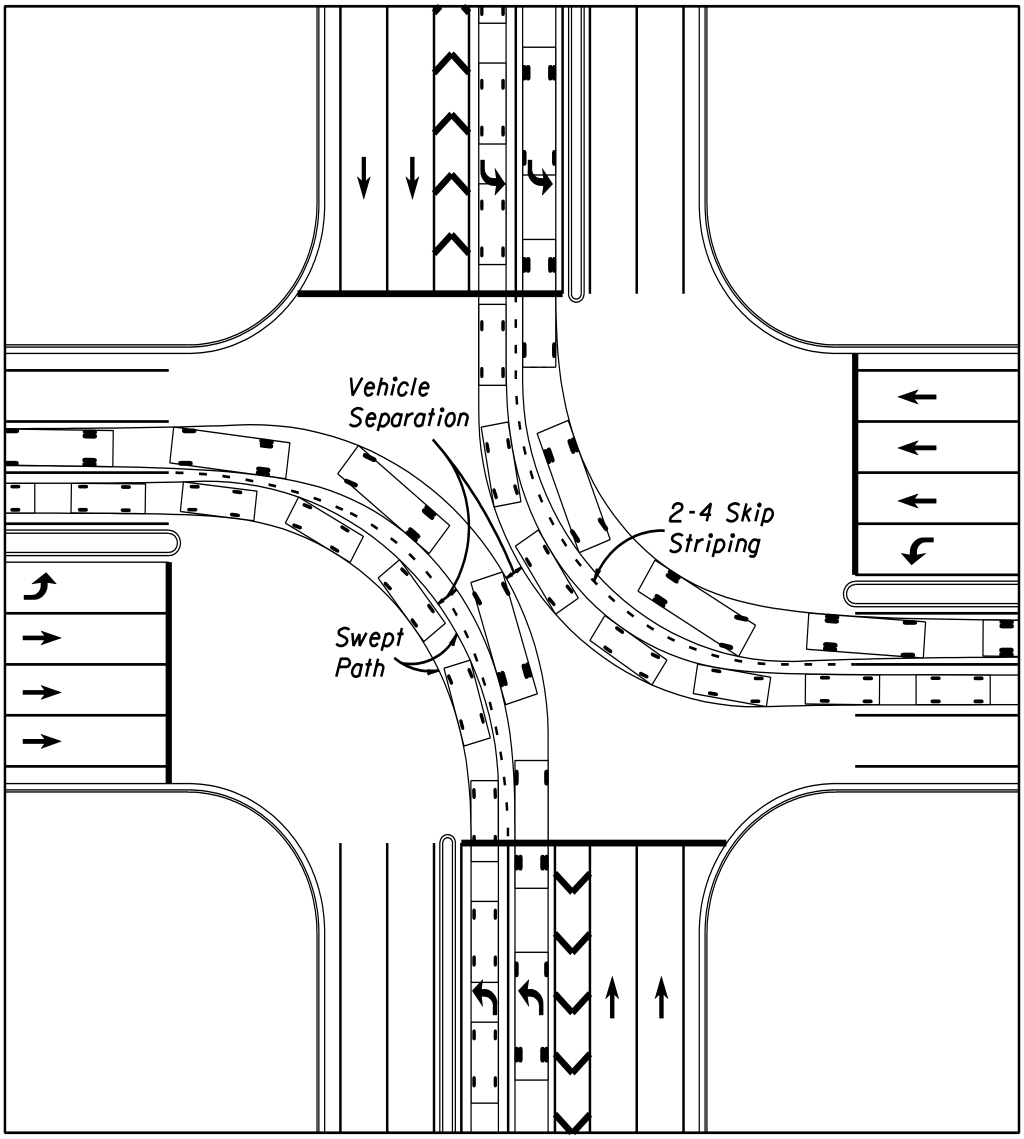


Figure 3-16 P and SU Design Vehicles Turning Simultaneously at Dual Left Turn Lanes

3.14 Sight Distance

Inadequate sight distance is a contributing factor in a large percentage of intersection crashes. The intersection design must provide sight distance for the driver to perceive potential conflicts and to traverse the intersection safely. The sight distance considered safe is directly related to vehicle speeds and to the resultant distances traversed during perception and reaction time and braking. The provisions for sight distance are sometimes limited by the roadway geometry and the nature and development of the area adjacent to the roadway. Sight distance requirements for roundabouts are presented in the Section 3.14.4 and ***Design Standards***, ***Index 546***.

Intersection Sight Distance requirements provided in this document are based on the ***2011 AASHTO Green Book*** and current Department requirements for use on the SHS. Sight distance requirements at at-grade intersections are based upon the following six different cases of intersection control:

Case A No control
Case B Stop control on the minor road
- Case B1 Left turn from the minor road
- Case B2 Right turn from the minor road
- Case B3 Crossing Maneuver from the minor road

Case C Yield control on the minor road
- Case C1 Crossing maneuver from the minor road
- Case C2 Left or right turn from the minor road

Case D Traffic signal control
Case E All-way stop control
Case F Left turns from the major road

3.14.1 Cases A and C

As mentioned in Section 3.2.3, all at-grade intersections on the SHS should be roundabouts, stop or signal controlled. Consequently, Case A should not be relevant to intersection design on the Florida Intrastate Highway System (FIHS). If applicable, however, the ***AASHTO Green Book*** should be consulted for guidance. Although the guidelines given for Case A generally consider collision avoidance by simply adjusting vehicle speeds, it is recommended that sight distance provision be based upon the vehicles being able to come to a complete stop. These distances are given in ***Table 3-14***. The road user should, at this recommended distance, have a clear view of the intersecting roadway as well as the stop sign or traffic signal at the intersection.

Table 3-14 Minimum Stopping Sight Distance [*PPM*]

MINIMUM STOPPING SIGHT DISTANCE (FEET) (For application of stopping sight distance, use an eye height of 3.5 feet and an object height of 0.5 feet above the road surface)		
DESIGN SPEED (mph)	GRADES OF 2% OR LESS	
	Interstate	All Other Facilities
15	----	80
20	----	115
25	----	155
30	----	200
35	----	250
40	----	305
45	----	360
50	----	425
55	570	495
60	645	570
65	730	645
70	820	730

ADJUSTMENT IN DISTANCE FOR GRADES GREATER THAN 2%

DESIGN SPEED (mph)	INCREASE IN LENGTH FOR DOWNGRADE (ft.)							DECREASE IN LENGTH FOR UPGRADE (ft.)						
	Grades							Grades						
	3%	4%	5%	6%	7%	8%	9%	3%	4%	5%	6%	7%	8%	9%
15	0	0	1	2	3	4	5	5	5	6	6	7	7	7
20	1	2	3	5	6	8	10	6	7	8	8	10	10	11
25	3	5	7	10	12	15	18	8	9	11	12	13	14	16
30	5	8	11	15	18	22	27	10	12	14	16	18	20	21
35	7	11	16	21	26	31	37	13	16	19	21	24	26	28
40	10	15	21	28	34	41	49	16	20	24	27	30	33	36
45	18	25	32	40	48	57	67	16	21	25	29	33	37	40
50	21	29	39	49	59	70	82	20	26	32	37	42	46	50
55	25	35	46	58	70	84	98	26	33	39	45	52	57	62
60	28	40	53	68	82	99	116	32	40	48	55	62	69	75
65	37	51	67	83	101	120	140	33	43	52	61	69	77	84
70	41	58	76	95	115	137	161	40	52	62	72	82	91	99

3.14.2 Case B

After a vehicle has stopped at an intersection, the driver must have sufficient sight distance to make a safe departure through the intersection area. The intersection design should provide adequate sight distances for each of the vehicle maneuvers permitted upon departure (i.e., crossing, left turn and right turn) from the stopped position. These maneuvers must be provided for, as well as the sight distances that must be provided for vehicles approaching on the major roadway from either the right or the left.

Distance "d" in ***Figure 3-18*** is the distance traveled by the respective vehicle on the major roadway traveling at design speed during the time required for the stopped vehicle to depart from its stopped position and either cross the intersection or to turn onto its desired leg of the major roadway. This value establishes one leg of the sight triangle. The leg of the stop-controlled road will be determined by the assumed location of the driver's eye. FDOT generally recommends that the assumed driver eye position be a minimum of 14.5 feet back from the edge of the traveled way (see ***Figure 3-17***). This distance can be adjusted when documented by a site specific field study. The third leg of the sight triangle is the actual sight line, which is the hypotenuse connecting the other two legs of the triangle. Thus, all sight obstructions shall be removed from within this area.

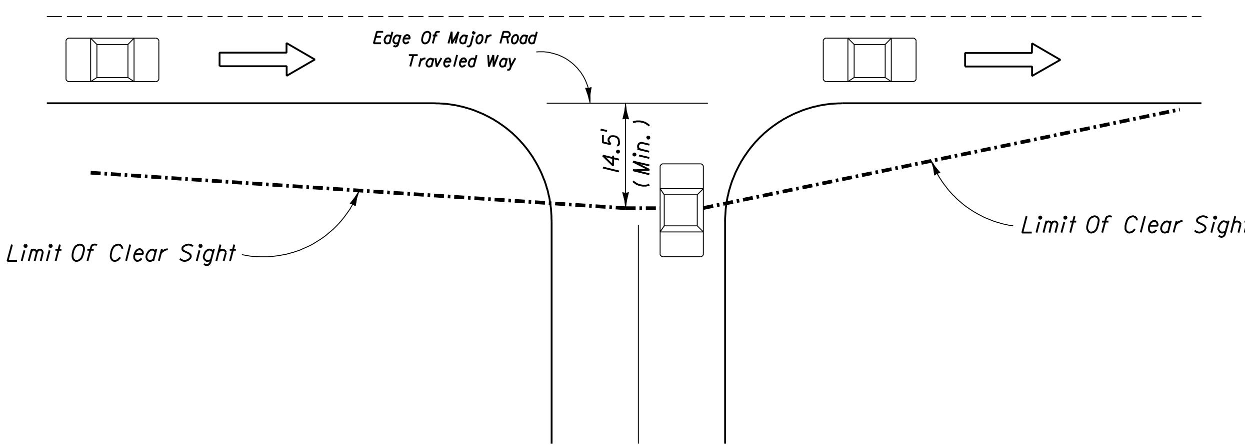


Figure 3-17 Origin of Clear Sight Line

Sight distance values have been summarized in ***Design Standards, Index 546*** (see ***Figure 3-18*** and ***Table 3-15***). These values apply to both rural and urban intersections under stop sign control or flashing beacon control. They were derived based on Case B of the ***AASHTO Green Book***, Chapter 9 and Department practices for channelized median openings (left turns from major roadways) [***PPM***]. These values also accommodate the Case F scenario.

Sight distance (d) is measured along the major roadway from the center of the intersecting roadway. It applies to normal and skewed intersections (intersecting angles between 60^o and 120^o) and where vertical and/or horizontal curves are present. Related distances d_L and d_r are measured from the centerline of the intersecting roadway to a point on the edge of the nearside outer traffic lane on the major roadway. Related distance d_m is measured from the centerline of the intersection to a point on the median clear zone limit for the far-side roadway of the major roadway [***PPM***].

Table 3-15 Minimum Sight Distance and Related Distances for 2-Lane Undivided, Multilane Undivided and Multilane Divided Roadways [*Design Standards*, Index 546]

Type Facility	Distance (feet)	Design Speed (mph)							
		30	35	40	45	50	55	60	65
Two-lane Undivided	d	335	390	445	500	555	610	665	720
	d_L	240	275	315	350	390	430	470	510
	d_r	150	175	200	225	250	275	300	325
Multilane Undivided	d	355	415	475	530	590	650	705	765
	d_L	250	295	335	375	415	460	500	540
	d_r	115	135	155	175	195	210	230	250
Multilane Divided	d	390	460	520	590	650	720	780	850
	d_L	280	330	370	420	460	510	550	600
	d_r	90	100	110	130	140	160	170	190
	d_m	320	380	430	480	530	590	640	700

NOTE: Additional tables are available in ***Design Standards, Index 546*** that address other configurations and design vehicles other than the passenger car.

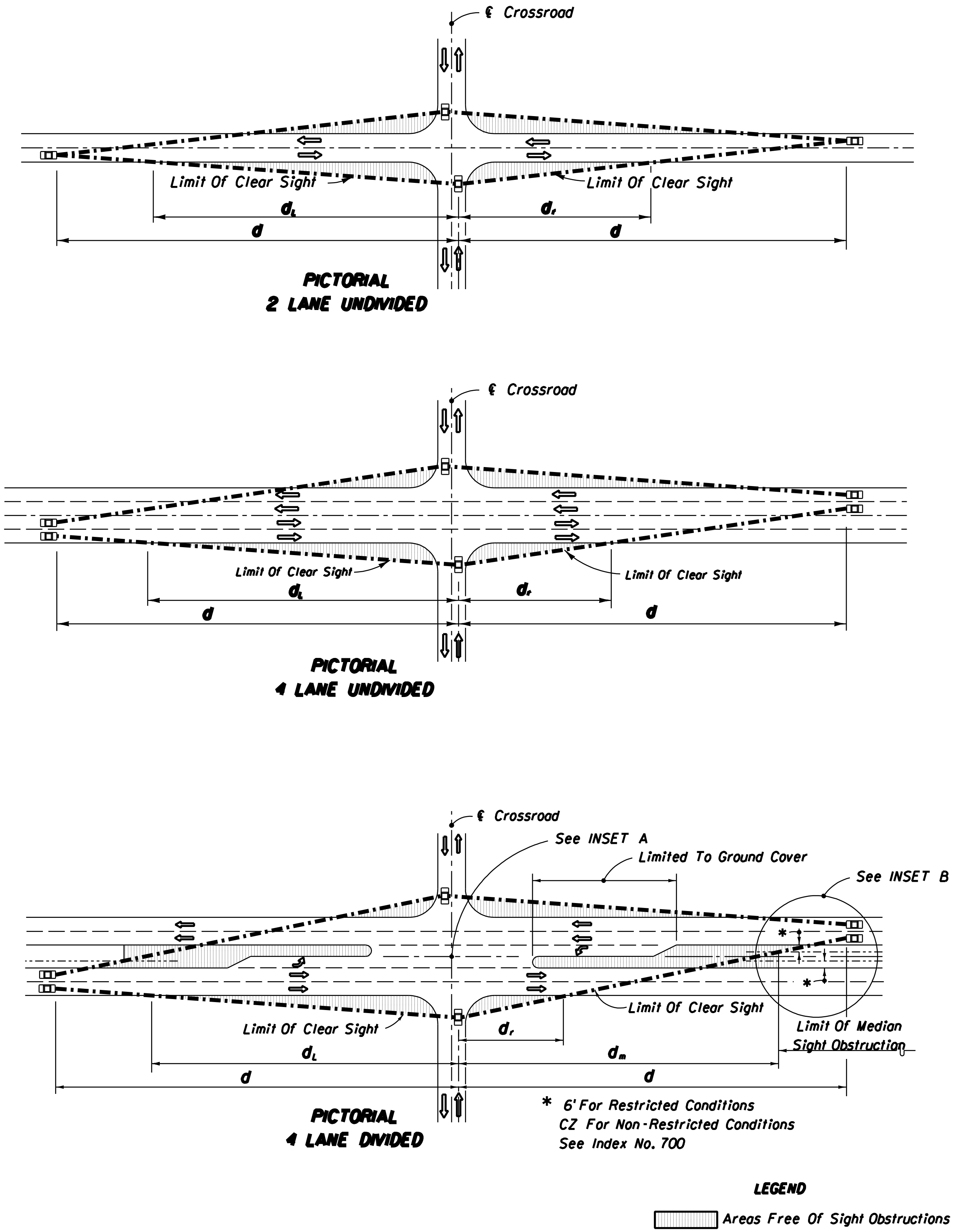


Figure 3-18 Sight Distance and Related Distances at Intersections

3.14.3 Case D & E

Due to a variety of standard operational characteristics associated with signal controlled intersections, the sight distances based on Case B procedures should be available to the driver. Unanticipated vehicle conflicts at signalized intersections, such as violation of the signal, turns on red, malfunction of the signal or use of the flashing red/yellow mode further substantiate the need for incorporation of Case B sight distances. If the proper distances cannot be obtained, other design features such as “no right-turn-on-red” may be necessary [***PPM***].

3.14.4 Roundabouts

Sight distances at roundabouts are required for several components: Stopping sight distance must be provided on the approach roadway to enable a stop at either the pedestrian crosswalk or at the yield line if crosswalks are omitted. Intersection sight distance (sight triangle) must be provided on the conflicting leg and within the circulating roadway in order for motorists to determine when to enter the roundabout. Stopping sight distance must also be provided for motorists to stop at the pedestrian crosswalk on exit from the roundabout. **Figure 3-17** depicts these sight distance components.

In most cases it is best to provide no more than the minimum required intersection sight distance on each approach. Excessive sight distance can lead to higher speeds that reduce the safety of the intersection for all road users (motorists, bicyclists, pedestrians). Intersection sight distance should be limited to a point 50 feet in advance of the yield line. If the approach leg of the sight distance triangle is greater than 50 feet, it may be advisable to add landscaping to restrict sight distance to the minimum required. Mounding and landscaping of the central island will make the roundabout more apparent to approaching traffic, and reduces headlight glare. Street lighting is required at all roundabouts for nighttime visibility.

Stopping sight distance is calculated using the following equation:

$$d = (1.468)(t)(V) + 1.087\frac{V^2}{a}$$

where

d = stopping sight distance, ft;
t = perception- brake reaction time, assumed to be 2.5 s;
V = initial speed, mph; and
A = driver deceleration, assumed to be 11.2 ft/s^2

Table 3-16 gives stopping sight distances computed from the above equations.

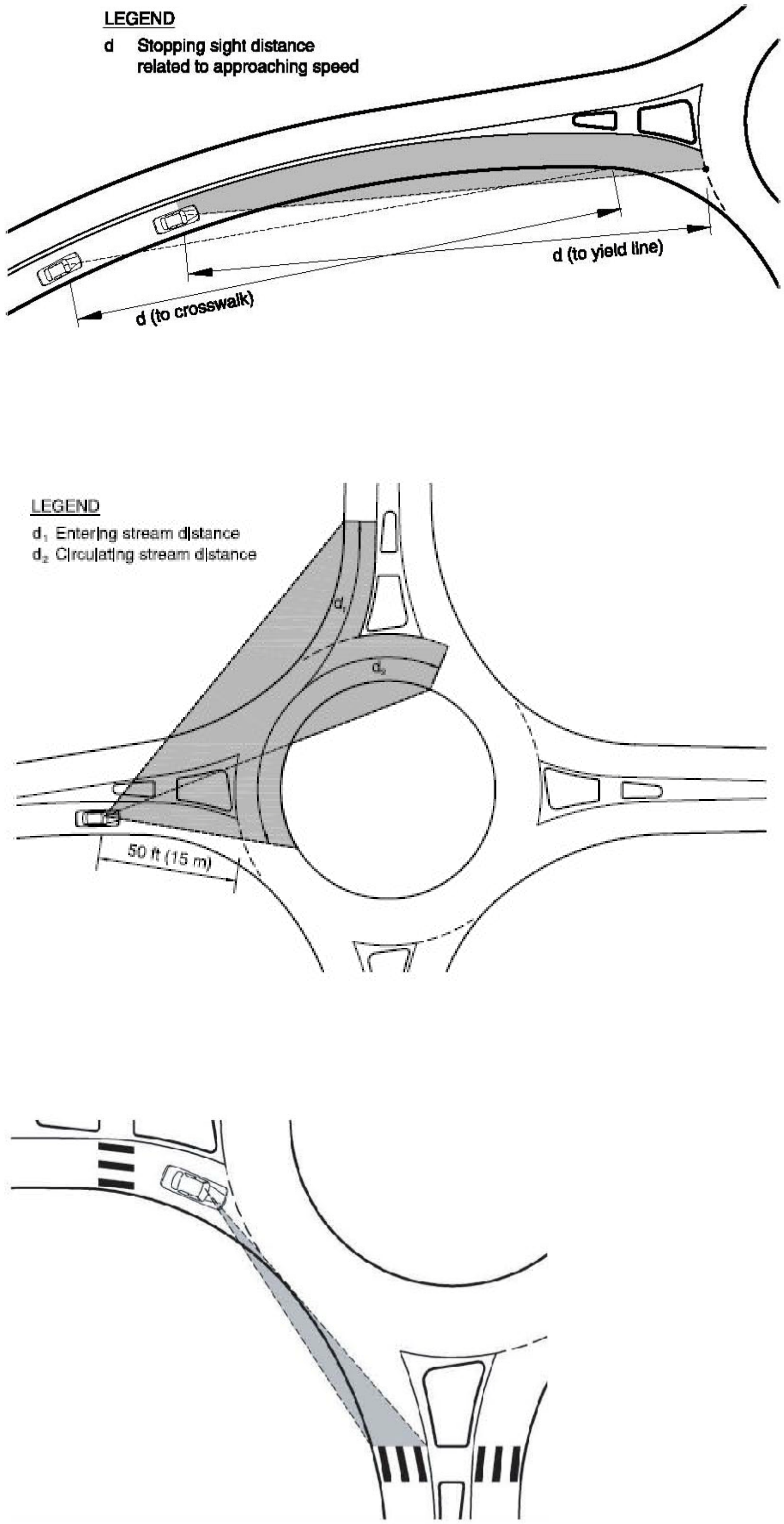


Figure 3-19 Roundabout Sight Distance

Table 3-16 Roundabout Stopping Sight Distance

Speed (mph)	Computed Distance ‘d’ (ft)
10	46.4
20	77.0
25	152.7
30	197.8
35	247.8
40	302.7
45	362.5
50	427.2
55	496.7

Intersection sight distance at a roundabout is primarily for the approaching vehicle to determine right-of-way. The approaching vehicle faces conflicting vehicles within the circulatory roadway and on the immediate upstream entry. The length of the conflicting leg is calculated using the following equations:

$$d_1 = (1.468)\left(V_{major,entering}\right)(t_c)$$

$$d_2 = (1.468)\left(V_{major,circulating}\right)(t_c)$$

where

d_1 = length of entering leg of sight triangle, ft;
d_2 = Length of circulating leg of sight triangle, ft;
V_{major} = design speed of conflicting movement, mph, discussed below; and
t_c = critical headway for entering the major road, s, equal to 5.0s.

Two conflicting traffic streams should be checked at each entry:

1. *Entering Stream*, which is composed of vehicles from the immediate upstream entry. The speed for this movement can be approximated by taking the theoretical entering (R_1) speed and the circulating (R_2) speed.
2. *Circulating Stream*, which is composed of vehicles that enter the roundabout prior to the immediate upstream entry. This speed can be approximated by taking the speed of left-turning vehicles.

Table 3-17 gives intersection sight distances based on the above equations.

Table 3-17 Roundabout Intersection Sight Distance

Conflicting Approach Speed (mph)	Computed Distance (ft)
10	73.4
15	110.1
20	146.8
25	183.5
30	220.2

3.14.5 Landscaping Considerations

Landscaping can have a significant impact on sight distance. The ***Design Standards, Index 546*** indicate that the corridor defined by the limits of clear sight is a restricted planting area. Drivers of vehicles on the intersecting roadway and vehicles on the major roadway must be able to see each other clearly throughout the limits of 'd'. If, in the Engineer's judgment landscaping interferes with the line of sight corridor prescribed by these standards the Engineer will direct rearrangement, relocation or elimination of plantings.

3.15 ACCESS MANAGEMENT

3.15.1 Florida Access Classification

Florida Statute, Section 335.18, SHS Access Management Act, of July 1988 requires that the State of Florida work with local governments in managing access on the SHS. ***Rule Chapter 14-97*** requires that all segments of the SHS be assigned an access classification with associated access standards. The standards are the basis for driveway permitting and the planning and development of FDOT construction projects.

The Florida access classification system consists of seven different access classes, (see ***Table 3-18)*** with Access Class 1 being used for limited access highways (i.e., freeways) and Access Classes 2 through 7 being used for controlled access highways (i.e., arterials). Controlled access highways are arranged from the most restrictive (Access Class 2) to the least restrictive (Access Class 7). These seven access classes are defined in [***Rule Chapter 14-97***].

Table 3-18 Florida Access Classes

A. Freeway Interchange Spacing

Access Class	Area Type	Segment Location	Interchange Spacing (miles)
1	Area Type 1	CBD & CBD Fringe for Cities in Urbanized Areas	1.0
	Area Type 2	Existing Urbanized Areas Other Than Area Type 1	2.0
	Area Type 3	Transitioning Urbanized Areas and Urban Areas Other Than Area Type 1 or 2	3.0
	Area Type 4	Rural Areas	6.0

B. Arterial Access Management Classifications & Standards

Access Class	Medians "Restrictive" physically prevent vehicle crossing. "Non-Restrictive" allow turns across at any point.	Connection Spacing (feet)		Median Opening Spacing (feet)		Signal Spacing (feet)
		>45 mph	≤45 mph	Directional	Full	
2	Restrictive with Access Roads	1320	660	1320	2640	2640
3	Restrictive	660	440	1320	2640	2640
4	Non-Restrictive	660	440			2640
5	Restrictive	440	245	660	*2640/1320	*2640/1320
6	Non-Restrictive	440	245			1320
7	Both	125	125	330	660	1320

∗ 2640 feet for >45 mph; 1320 feet for ≤ 45 mph

3.15.2 Driveway Design

Driveway design shall be done in accordance with ***Design Standards, Index 515*** and ***516***. ***Index 515*** identifies the elements of a driveway and prescribes widths, angles, radii etc. for various traffic volume levels. It also provides profiles and construction details for driveways. ***Index 516*** prescribes requirements for driveways in resurfacing projects. Driveways should not be permitted in the circulatory roadway of a roundabout unless it has sufficient volume to consider it as an additional leg of the intersection.

Be aware that due to high traffic volumes or large vehicles expected on a driveway connection, it may be desirable to design the driveway more like a full intersection. In this case, the criteria in ***Design Standards, Index 515*** and ***516*** may not apply.

3.15.3 Median Design

The ***Median Handbook*** provides a complete technical guide to median decisions. The topics covered in this reference include concepts and definitions, Departmental policy on medians and median openings; crash comparison and public opinion related to medians; median opening design and placement guidelines and operational considerations such as pavement markings, landscaping, etc.

Chapter 4

Signalization

Tables

Figures

4 SIGNALIZATION

Signalization design includes both the physical installation and the signal operating plan. It is important that both be compatible with each other and with the geometric configuration. All of the desirable objectives of an intersection design presented in Chapter 1 must be considered and sometimes weighed against each other. This chapter sets forth the principles and criteria for signal design that are recognized by the FDOT, and identifies the design methodology that is commonly used throughout the state.

4.1 SIGNIFICANT REFERENCES

The following reference documents govern the design of signalization features at intersections in Florida. The references indicated in *italics* were described in Chapter 1.

1. The Manual on Uniform Traffic Control Devices (***MUTCD)*** establishes nationwide standards that promote uniformity to facilitate driver comprehension of traffic control devices. Part 4 of the ***MUTCD*** covers signals. Florida Statutes Chapter 316, State Uniform Traffic Control promulgates required devices and motorist responsibilities at traffic control signals. References to ***MUTCD*** requirements pertaining to specific topics are provided in boxes throughout this chapter. The current version of the ***MUTCD*** is available in printed form and on the Internet in electronic form at: http://mutcd.fhwa.dot.gov/kno_2009r1r2.htm

MUTCD Part 4 Sections

A. General
B. Traffic Control Signals
C. Signal Needs Studies
D. Signal Features
E. Pedestrian Control Features
F. Pedestrian Hybrid Beacons
G. Signals and Hybrid Beacons for Emergency Vehicle Access
H. Signals for One-Lane, Two-Way Facilities
I. Signals for Freeway Entrance Ramps
J. Traffic Control For Movable Bridges
K. Signals at Toll Plazas
L. Flashing Beacons
M. Lane-Use Control Signals
N. In-Roadway Lights

2. The Highway Capacity Manual (***HCM***) prescribes a procedure for evaluating the performance of a signalized intersection as a function of the intersection configuration, the signal operation and the traffic conditions. The current version of the ***HCM*** is known as ***HCM 2010***. Specialized software has been developed to automate HCM design procedures.

3. The ***PPM,*** Volume I, Section 7.4 outlines the general requirements affecting signal design. The following topics are covered in the ***PPM***:

a. Design Criteria;
b. Certification and specialty items;
c. Stop line location;
d. Controller assemblies;
e. Left turn treatments;
f. Signal preemption;
g. Lane configuration;
h. Signal Loops;
i. Grounding and Electrical Bonding;
j. Wind Loading (Chapter 29);
k. Foundation Criteria;
l. Mast arm supports;
m. Traffic Signal Traffic Coordination;
n. LED Light Sources;
o. Pedestrian Countdown Signal Applications;
p. Number of Signal Heads for Through Lanes;
q. Backplates; and,
r. Span Wire Assemblies.

Many of the same topics are also covered in this chapter. The guidelines presented here are consistent with the requirements of ***PPM***, with additional details provided to assist the designer.

4. FDOT ***Standard Specifications for Road and Bridge Construction (FDOT Specifications)***: includes a section covering all traffic control signal equipment and materials used in the construction of signalized intersections.

5. The FDOT ***Design Standards*** set forth the following requirements that apply specifically to signal design:

Index 17700 Pull, Splice and Junction Box
Index 17721 Conduit Installation Details
Index 17723 Steel Strain Pole
Index 17725 Concrete Poles
Index 17727 Signal Cable and Span Wire Installation Details
Index 17733 Aerial Interconnect
Index 17736 Electric Power Service
Index 11743 Standard Mast Arm Assemblies
Index 17745 Mast Arm Assemblies
Index 17748 Free-Swinging, Internally-Illuminated Street Sign Assemblies
Index 17764 Pedestrian Control Signal Installation Details
Index 17781 Vehicle Loop Installation Details
Index 17784 Pedestrian Detector Assembly Installation Details
Index 17841 Cabinet Installation Details
Index 17870 Standard Signal Operating Plans
Index 17881 Advance Warning for R/R Crossing
Index 17882 Railroad Grade Crossing Traffic Control Devices
Index 17890 Traffic Control Devices for Movable Span Bridge Signals

Index 17870 deals with the operational aspects of signalization plans. All others deal with items related to physical configuration and installation.

6. The FDOT ***MUTS*** prescribes the procedures for conducting several studies related to signal design including:

    a. Traffic Signal Study Procedure
    b. Signal warrant studies;
    c. Vehicular and pedestrian counts and characteristics;
    d. Collision and condition diagrams;
    e. Performance evaluation studies;
    f. Traffic and pedestrian flow characteristics; and
    g. Vehicle spot speed studies.

7. The FDOT ***Traffic Engineering Manual (TEM)*** contains sections covering:

    a. Flashing signals and beacons;
    b. Left-turn treatments;
    c. Standardization of yellow and all red intervals;
    d. Traffic signal studies;
    e. Emergency traffic control signals;
    f. Pedestrian signals;
    g. Computer models for traffic engineering and ITS analysis and design; and
    h. Certification and approval of traffic control signals and devices.

8. ITE ***Traffic Engineering Handbook*** (2009)

9. AASHTO ***Guide for the Development of Bicycle Facilities*** *(1999)* provides guidelines for:

    a. Intersection clearance times for bicycles;
    b. Signalization at intersections with shared use paths and trails;
    c. Location of push-button detectors for cyclists;
    d. Visibility of signal indications; and
    e. Bicycle detection for traffic-actuated control.

9. ***Trail Intersection Handbook*** *(1996)* offers guidelines for the treatment of bicycles and pedestrians where intersections with trails are signalized.

General reference material describing signalization concepts, design tools, controller functions, etc. may be found in the following publications:

1. *National Electrical Manufacturers' Association (NEMA)* Standards for Traffic-actuated Controllers: This publication describes the physical and functional requirements of signal controllers. It creates a limited degree of interchangeability between the products of different manufacturers. Two standards, TS-1 and TS-2, are prescribed. TS-1 dates back to the 1970's but still applies to most of the equipment in current use. TS-2 incorporates contemporary computer and communications technology.

2. *Methodology for Optimizing Signal Timing (M|O|S|T)* is a comprehensive reference document available from McTrans. This document covers all aspects of signal timing design, implementation and performance evaluation.

3. *Manual of Traffic Signal Design (ITE)* deals with all aspects of signal design, including physical and operational characteristics, equipment, displays, installation, etc.

4. A course in Basic Traffic Signal Operations is offered periodically through the Florida Engineering Education Delivery System (FEEDS).

Several software products are available to perform computations for signal timing design and performance evaluation. These tools will be described later in this chapter.

4.2 TRAFFIC SIGNAL CONTROLLER OPERATION

In theory, a signal controller may be either pre-timed or traffic-actuated. Pre-timed controllers display fixed phase durations that repeat from cycle to cycle. Traffic-actuated controllers use vehicle presence information from detectors to generate phase durations appropriate to accommodate the demand on each cycle.

As a practical matter modern signal controllers are made up of a group of phase modules, each of which controls one or more traffic movements. Each phase module is capable of traffic-actuated operation. Pre-timed phases, where required, are created by imposing constraints on the traffic-actuated features to force a constant duration.

The basic principle upon which traffic-actuated controllers respond to current demand is very simple. The green display is held at first for a prescribed minimum length of time. After the minimum green time has elapsed, the strategy is to terminate the phase upon satisfaction of the queue of vehicles accumulated on the previous red phase. Maintaining the green display after the queue has been satisfied creates wasted time that could be better used to serve other movements.

The controller uses the length of the gaps between vehicles observed at the detector to determine when the queue has been satisfied. As soon as a gap of a specified length occurs, the controller invokes the phase termination strategy. This promotes the display of green intervals that are appropriate to the vehicular demand on each cycle. During periods of light traffic, phases will be short. As traffic volumes increase, so will the phase durations. Of course, it is also necessary to specify the maximum length of each phase to keep the cycle length to a reasonable level and to stabilize the operation in the event of stalled vehicles or detector failure.

Each phase module therefore requires the following parameters to be specified for basic traffic-actuated operation:

a. The minimum time for which the green interval must be displayed regardless of traffic requirements;

b. The maximum time for which the green interval may be displayed if there is demand from competing movements and

c. The length of a gap between successive vehicles moving on the green phase that will cause the phase to be terminated.

Basic traffic-actuated operation uses fixed values for both the minimum green time and the allowable gap. A more complex traffic-actuated mode causes one or both of these values to vary with traffic demand. The industry has attached the term “volume-density” to these advanced features. Volume-density control is favored regionally for use at isolated intersections with high approach speeds.

4.2.1 Signal Phasing

The signal sequence is controlled by a group of interconnected phase modules, each of which generates the commands to control a single green, yellow, red, walk and don’t walk display, with proper transitions between displays. Each module accepts vehicle detector inputs that indicate whether or not a vehicle is present on the approach controlled by the phase. Only one detector input is recognized per phase.

Vehicular presence during the red interval will place a demand for a green signal to be displayed. The detector is ignored during the display of the minimum green interval, but after this interval has expired, the presence of a vehicle will cause the phase to be extended until a gap of the prescribed length has been encountered.

Each phase module also has a single input for pedestrian demand. If vehicular demand is received without pedestrian demand, the phase timing will be as described above. In this case the pedestrian signal will display a constant “don’t walk” indication. If pedestrian demand is received, with or without vehicular demand, the phase module will display a “walk” interval of the prescribed duration followed by a “don’t walk” interval. After the pedestrian timing has been satisfied, the normal phase extension process will be invoked until a gap in the traffic causes the phase to terminate.

The signal controller consists of a group of these phase modules interconnected to pass control from one to another in an orderly fashion. Two examples of the representation of signal phasing, as presented in the ***Design Standards, Index 17870***, are shown in ***Figure 4-1*** and ***Figure 4-2***. In the simplest form of operation, only one phase module will assume control at any time. The rest will remain in the red state, but will register a demand for their turn whenever they detect vehicular or pedestrian presence.

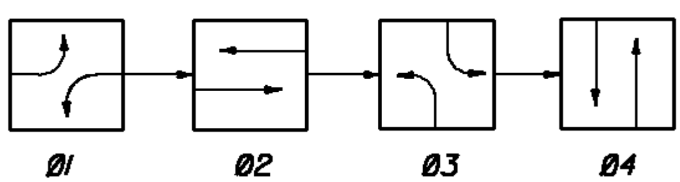


Figure 4-1 Phasing Diagram for Single Ring Sequential Signal Phasing

This simple sequence is known as *single-ring sequential* operation. In its most elementary form, only two phase modules will be involved in the sequence, with each controlling an opposing pair of approaches (or non-conflicting movements). Additional modules may be added to a single ring operation to provide exclusive phases for left turns or other minor movements. An example of single ring operation is presented in ***Figure 4-1***, with four phases displayed sequentially as the terminology suggests. Note that each phase module controls two approaches that move simultaneously.

Most traffic-actuated intersections in Florida use a more complex scheme known as *dual-ring concurrent* operation, which is depicted in ***Figure 4-2***. Note that Phases 2 and 5 in the phasing diagram offer two alternatives, only one of which may be displayed on any cycle.

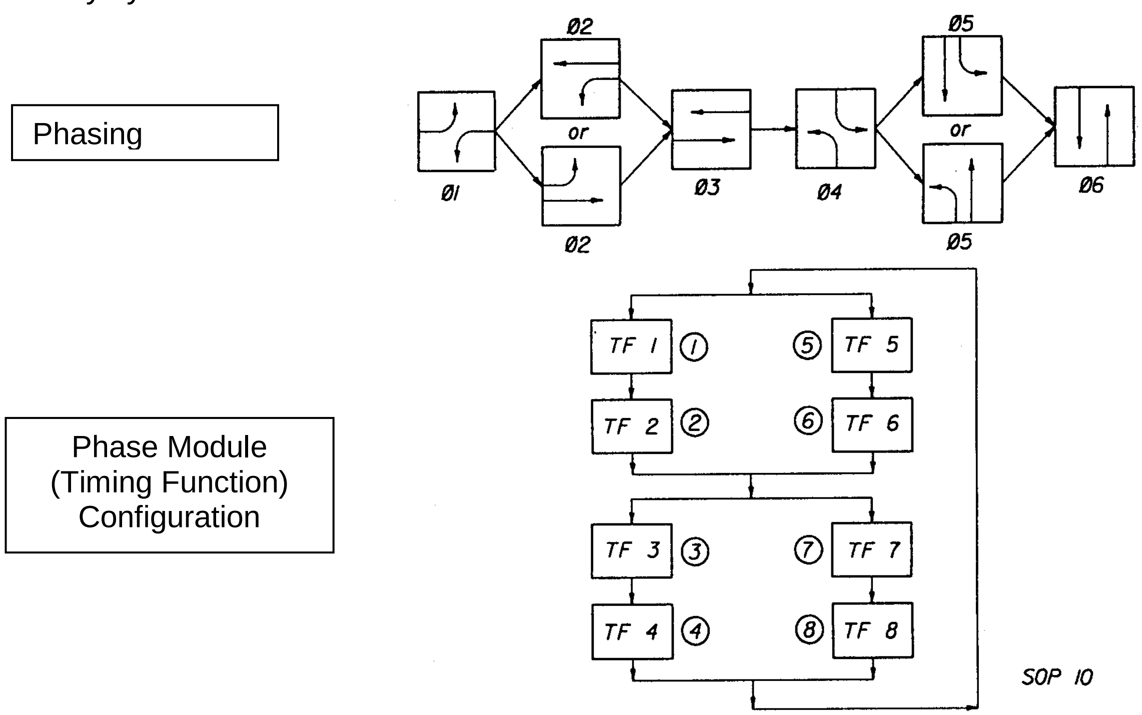


Figure 4-2 Representation of Dual Ring Concurrent Signal Phasing

Dual-ring concurrent phasing is generally more efficient than single-ring sequential phasing because it allows the available green time to be distributed with less “slack,” i.e., unused green time. ***Design Standards***, ***Index 17870*** refers to the phase modules as *Timing Functions* (TF). TF 1-4 constitutes Ring 1 and TF 5-8 constitutes Ring 2. Each timing function controls only one movement. Since two movements can proceed simultaneously without conflict, two of the timing functions (one from each ring) will always have control simultaneously. This process is transparent to the road user.

4.2.2 Signal Timing

Signal timing involves the determination of the appropriate cycle length (i.e., the time required to execute a complete sequence of phases) and apportionment of time among competing movements and phases. The timing apportionment is constrained by minimum green times that must be imposed to provide for pedestrians and to ensure that motorist expectancy is not violated.

Within these constraints, there are three objectives that may be chosen to optimize the timing design:

a. Equalization of the degree of saturation (volume/capacity ratio) among competing movements: This design is usually associated with simple manual methods of signal timing because it is the only approach that can be implemented productively without computer software. It is also the starting point for many software-based procedures.

b. Minimization of the aggregate delay to all vehicles using the intersection: This is the only objective that could be considered as a true optimization. The others are based on equalizing driver-perceived disutility. Some procedures consider stops, fuel consumption and operating cost in addition to delay in the optimization process.

c. Equalization of control delay among competing movements: While this is not a true optimization, it is popular because the ***HCM*** defines level of service in terms of delay to each vehicle. Therefore the equalization of delay results in a more equitable assignment of intersection level of service among all road users.

The software products used for designing signal timing plans are identified in Chapter 4 and the specific parameters to be determined by the signal timing process are discussed.

4.3 DESIGN REQUIREMENTS

Before describing the design procedure itself, a few key concepts with which the designer must be familiar will be addressed.

> The selection and use of traffic control signals should be based on an engineering study of roadway, traffic, and other conditions.
>
> ***{MUTCD}***

4.3.1 Signal Controller Cabinet Elements

The traffic controller cabinet contains several functional modules that require the attention of the designer, including:

1. The controller itself, which requires all operating parameters to be specified;
2. A conflict monitor, which requires conflicting phases to be identified;
3. A coordination unit, which may require parameter settings;
4. A network switch, which requires protocol settings; and,
5. Vehicle detectors, which may require parameter settings.

Several industry standards now exist for signal controllers and this area is evolving to keep pace with the demands of intelligent transportation systems. The choice of controller standard for a given intersection will be determined by local policies and is beyond the scope of this document.

4.3.2 Design Products and Prerequisites

The product of the signalization design process is generally a plans set prepared in accordance with Volume II of the ***PPM***. In the case of simple operational modifications or upgrades to existing signals, the design product may take the form of a work order issued by the operating agency.

The general intersection design process and data requirements were discussed in Chapter 2. In this chapter, the process will be applied specifically to signalization. It will be assumed that the following information is available, either as field data or as a result of decisions that have already been made:

> The signalization for each intersection should be individually designed.
> ***{PPM}***

1. The intersection in question will be signalized.
2. The movements and modes to be accommodated have been identified and the traffic volume, composition and speed are known for each movement. Note: The existing speed limits should be reviewed and modified if necessary as a part of the intersection design.
3. The intersection configuration (cross section, angle of intersection, channelization, number and width of lanes and lane use) has already been established.
4. Accommodations for bicycles and pedestrians, including sidewalks, bicycle lanes and shared use paths have been identified.
5. The design controls for coordinated operation of the intersection are known, including cycle length constraints, allowable phasing alternatives, distances to adjacent intersections and number of timing plans to be developed.
6. The location and utilization of access points near the intersection, including median openings, has been identified.

7. All special requirements affecting the signalization, including railroad, drawbridge and emergency vehicle preemption have been specified.

4.3.3 Signalization Design Decisions and Parameters

The key decisions affecting the signal design include:

1. Location of crosswalks and determination of pedestrian signal display requirements;
2. Location of stop lines for all movements that must stop at the intersection;
3. Selection of left turn treatments;
4. Selection of right turn treatments;
5. Determination of the required lengths for turning bays;
6. Selection of controller type;
7. Selection of the phasing plan;
8. Assignment of movements to timing functions;
9. Development of the timing plan(s) for arterial coordination;
10. Determination of detector locations;
11. Location and configuration of the controller and cabinet;
12. Location and configuration of all signal displays;
13. Location of equipment required to support the signal displays; and
14. Wiring, conduit and pull boxes.

The above list is presented in the likely order in which the decisions must be made. It is important to note, however, that some decisions may not be entirely independent of others and therefore some iteration between decisions may be required. Requirements and guidelines for each of the key signalization design decisions will now be presented.

4.4 LOCATION OF CROSSWALKS AND DETERMINATION OF PEDESTRIAN SIGNAL DISPLAY REQUIREMENTS

At this stage of the design, the location of the crosswalks is the primary concern, because of their influence on the stop line location. Chapter 5 will cover the detailed requirements of the crosswalk as a pavement marking feature.

> The design and operation of traffic control signals shall take into consideration the needs of pedestrian as well as vehicular traffic. If engineering judgment indicates the need for provisions for a given pedestrian movement, signal faces conveniently visible to pedestrians shall be provided by pedestrian signal heads.
>
> ***{MUTCD}***

4.4.1 Need for Crosswalks

Each approach to the intersection presents a potential need for a pedestrian crosswalk. Crosswalks should be incorporated into the signal design for all approaches unless the crosswalk location is not accessible to pedestrians or a decision has been made to prohibit pedestrian crossing.

> The first step in the design process should be to locate crosswalks and stop lines properly.
>
> ***{PPM}***

Pedestrian crossing should not be prohibited unless it cannot be accomplished safely. If pedestrian crossing is prohibited across a specific approach, the prohibition should be made clear in the design of the intersection. Alternative provisions must be made for the pedestrians who would normally cross that approach.

> Crosswalks should be marked at all intersections where there is substantial conflict between vehicular and pedestrian movements.
>
> ***{MUTCD}***

The signalization design should be such that it accommodates all modes of travel. Some compromises may be necessary when all requirements cannot be accommodated within the capacity of an existing intersection. Prohibition of pedestrian crossing in a specific crosswalk to achieve vehicular capacity goals should be considered as a last resort and only when the pedestrians may be accommodated safely by other means.

> Where it is desired to prohibit certain pedestrian movements at a traffic control signal, a NO PEDESTRIAN CROSSING sign may be used.
>
> ***{MUTCD}***

4.4.2 Need for Pedestrian Signals

The box at the right summarizes the conditions requiring pedestrian signals as set forth in the ***MUTCD***. There are several other conditions that also suggest a need for pedestrian signals.

Pedestrian signals shall be installed:
1. When the signal is installed under the pedestrian or school crossing warrant
2. When an exclusive pedestrian phase is provided
3. When multi-phase vehicular indications (as with split-phasing) would tend to confuse or cause conflicts with pedestrians guided only by vehicular indications
4. At established school crossings at any signalized location

{MUTCD}

1. When it is necessary to assist pedestrians in deciding when to begin crossing the road;
2. When engineering judgment determines that pedestrian signal heads are justified to minimize vehicle-pedestrian conflicts;
3. When pedestrians are expected to cross different portions of the street (such as to or from a median) on different phases;
4. When vehicular signal indications are not visible to pedestrians using the crosswalk
5. When protected left or right turn phases are included in the signal sequence;
6. When the crash history indicates a hazard that could be mitigated by pedestrian signals;
7. When the crosswalk is a part of an established pedestrian or bicycle corridor;
8. When the crosswalk is used by people with special needs; or
9. When an abnormal intersection configuration (skewed, multi-legged, etc.) exists.

The sum total of all of these conditions suggests that pedestrian signals should be installed at nearly all signalized crosswalks. When pedestrian signals are not included in the design, it is important that the minimum green time displayed to all approaches be adequate to accommodate pedestrian crossing requirements. The determination of minimum green times is discussed later in this chapter.

Pedestrian phase requirements:
1. Indications must be visible to pedestrians
2. Delays must not be excessive
3. An adequate crossing interval must be provided
4. Pedestrian crossing may be prohibited

{MUTCD}

4.4.3 Need for Protected or Exclusive Pedestrian Phases

It is necessary that the decision on the need for protected or exclusive pedestrian phases be made at this stage of the design process because of its effect on crosswalk and stop line location, operations, and intersection capacity. Exclusive pedestrian phases prohibit all vehicular movements to allow pedestrians to cross simultaneously in all directions. They are not often used because they normally cause a substantial increase in delay to both vehicles and pedestrians. There are, however, conditions of high pedestrian volumes or high volumes of right-turning traffic that make it necessary to separate the vehicular and pedestrian movements entirely.

Protected pedestrian phases provide for pedestrian crossing with no conflict from turning vehicles. When pedestrians proceed concurrently with a through movement, they are normally in conflict with right turns and permitted left turns from the same phase. Elimination of the pedestrian conflict requires either the prohibition of (or an alternate phasing accommodation for) the conflicting turns. Both the left and right turns may be accommodated on protected phases as long as sufficient capacity is available. It is important to keep in mind that protected turning phases require exclusive turning lanes that could increase pedestrian crossing distances.

A detailed evaluation of the need for and feasibility of exclusive pedestrian phases generally requires the use of one or more of the traffic operations models described later in this chapter.

4.5 LOCATION OF STOP LINES

The stop line location is heavily influenced by the location of the crosswalks. A stop line that is not properly located invites violation by the motorist.

> Stop lines, where used, should be placed 4 ft. in advance of and parallel to the nearest crosswalk line.
>
> ***{MUTCD}***

If no crosswalk, either marked or unmarked, is present on a given approach, the stop line should be located at the point where vehicles are expected to stop, typically 4 feet from where a crosswalk would be located if it were present. This point could also be influenced by left turning traffic approaching from the right. There is sometimes a tendency for the motorist to stop too close to the intersection, thereby obstructing the turning paths of left-turning vehicles. Proper location of the stop line may help to correct this problem.

> At least one and preferably both of the two minimum primary signal faces required for the through movement shall be located between 40 and 180 ft. from the stop line.
>
> ***{MUTCD}***

The ***MUTCD*** places constraints on the minimum and maximum distance between the stop line and the signal face in the interest of signal visibility. A more detailed discussion of the ***MUTCD*** requirements is presented in Section 4.15 of this chapter. The proper location of the stop line should govern the signal head placement. Stop lines should not be displaced from their proper position to accommodate the existing or proposed location of the signal displays. Sight distance restrictions should also be taken into account in locating stop lines.

When two streets intersect diagonally, the stop line location becomes more difficult. The alternatives are to paint a continuous diagonal stop line or a staggered stop line perpendicular to each lane. Preferences with respect to these alternatives vary among jurisdictions. It is generally agreed that mast arms that are used for signal mounting should be parallel to the stop bar. This consideration can affect both the signal mounting method and the stop line location.

4.6 SELECTION OF LEFT TURN TREATMENTS

Left turn treatments are defined in terms of:

1. The number of exclusive and optional lanes assigned to left-turning traffic;
2. The degree of protection from opposing through traffic provided by the signal phasing and
3. The position of a protected phase in the signal sequence with respect to the through phase.

Each of these design decisions will be discussed separately.

4.6.1 Definitions

In the following discussion, lanes that accommodate left turns only will be referred to as "exclusive" and those that accommodate left turns in combination with other movements will be denoted as "shared."

Left turns facing a solid green signal may proceed, but must yield to oncoming traffic. This operation may not provide an adequate level of capacity or safety and it is often necessary to provide protection for left turning movements from the oncoming traffic. This introduces a protected phase into the signal sequence to provide a green arrow display for left turns. Left turns that receive a green arrow display at some point in the sequence are defined by the ***HCM*** as "protected left turns." Left turns that are allowed to proceed, yielding to opposing traffic, are defined by the ***HCM*** as "permitted left turns."

If it is determined that a specific left turn requires a protected phase, it must also be decided whether or not the left turn also should be allowed to proceed during the permitted phase. The situation in which a left turn may proceed only on the protected phase is defined by common usage in Florida as "protected" or "protected only." The situation in which left turns may proceed on both a protected and permitted phase are commonly described as:

1. *Protected/permitted:* in which the permitted phase follows the protected phase and
2. *Permitted/protected:* in which the protected phase follows the permitted phase.

These definitions summarize the design choices that must be made with respect to left turn protection at traffic signals. Unfortunately, a consistent and universal terminology has not yet emerged in the traffic engineering vocabulary and other designations such as "permissive," "exclusive," and "restrictive" will be found in the literature. Furthermore, the terms are often applied in a somewhat loose and inconsistent manner. It should be noted that the ***MUTCD*** now describes the left turn treatment choices in terms of four "modes":

1. The "permissive only" mode;
2. The "protected only" mode;
3. The "protected/permissive" mode and
4. The "variable left turn" mode. This last mode has been introduced to describe a condition in which left turn treatment changes throughout the day.

It is essential that a set of left turn protection terms be established and applied consistently in this document. In the remaining discussion, the terms "protected" and "permitted" left turns will be adopted to describe the manner in which left turns are accommodated. Phasing schemes that involve a combination of permitted and protected treatment for a specific left turn movement will be denoted as "protected-permitted" with no order of phases implied. When it is necessary to specify a phase order, the terms "leading" and "lagging" will precede the treatment identification. So, for example, the left turn treatment for a specific phase could be described as "leading protected," "lagging protected-permitted," etc. Where it is necessary to distinguish clearly between protected and protected-permitted phasing, the term "protected only" will be used. A permitted left turn interval may be indicated by a green ball or flashing yellow arrow.

4.6.2 Need for Exclusive Left Turn Lanes

Left turns may be made from shared lanes after yielding to the opposing through traffic; however, the capacity of a shared lane is somewhat limited. The ***HCM*** provides a procedure for assessing the capacity of both shared and exclusive lanes under signal control.

The operational advantage of an exclusive lane is quite clear from a capacity perspective. The disadvantages apply primarily to pedestrians in the form of increased exposure in the crosswalk and reduced refuge area on the median. On roadways without medians exclusive left turn lanes must be created by widening the roadway. This can cause alignment problems as well as additional hazards or inconvenience for pedestrians.

As a general rule, a single left turn should be considered when the left turn volume exceeds 100 vehicles per hour. A double left turn lane becomes advisable when the left turn volume exceeds 300 vehicles per hour. Exclusive left turn lanes are normally required when protected left turn movements are provided in the signal phasing. Without a protected phase, signalized approaches with adequate median width should normally provide exclusive left turn lanes unless left turning volumes are negligible or unless the addition of a left turn lane would negatively impact pedestrians. When no medians are present and a widening of the approach would be required, the decision to add an exclusive left turn lane should be based on a demonstrated operational or safety related need. Pedestrian impacts that might result from a loss of sidewalk area or utility pole conflicts should also be considered.

When left turning volumes are high, multiple exclusive left turn lanes may be required to provide adequate capacity. Double left turn lanes should be considered when a capacity analysis suggests that overall intersection performance could be improved. Triple left turn lanes have performed effectively at numerous intersections in Florida. They do, however require more justification and attention to the design details.

Proper attention must be paid to accommodating traffic in multiple left turn lanes as it leaves the intersection. The exit roadway must have enough lanes to accommodate the left turns and pedestrian crosswalks should be clearly marked. Vehicle tracking must be closely scrutinized to avoid an overlap with conflicting movements. Pedestrian signals should always be used for any crosswalk in which pedestrians will encounter protected left turns, regardless of the number of lanes.

4.6.3 Need for Left Turn Protection

There are no numerical warrants that have been officially adopted for determining when left turns require protected phasing. The primary determinants of the need for protection are the left turn volume and the degree of difficulty in executing the left turn through the opposing traffic. A cross-product threshold technique is used by some agencies as a guideline. The cross product is defined as the product of the left turning volume and the volume of opposing traffic. The guidelines provided by the ***HCM*** for planning purposes suggest that protection for left turns should be considered when the left turn demand is greater than 240 vehicles per hour, and the cross product exceeds 50,000, 90,000 and 110,000 for one, two and three lanes of opposing traffic, respectively.

A detailed capacity analysis of the operation of each movement with and without left turn protection can provide an indication of the adequacy of both phasing alternatives from a capacity point of view. The addition of a protected left turn phase for a specific left turn movement will take green time away from all other movements, thereby reducing their capacity. On the other hand, failure to provide a protected phase may leave the left turn in question without adequate capacity. A design dilemma arises when these two conditions occur simultaneously (i.e., a left turn will be over capacity without a protected phase, but the whole intersection will be over capacity if a protected phase is added).

> Protected-permissive phasing should be used except when there is a compelling reason not to.
>
> ***{TEM}***

Capacity analysis is a useful tool; however, safety concerns are generally raised by the public and are sometimes indicated by the intersection's safety performance at volume levels well below the capacity of the left-turning movement. The safety performance of an intersection should be given more weight than capacity or other performance measures in determining the need for left turn protection. Area demographics should also be considered in assessing potential safety impacts.

Note that the feasibility of providing left turn protection depends to some extent on the lane configuration. In general, exclusive left turn lanes are required to implement left turn protection satisfactorily. It is assumed in that the availability of an exclusive left turn lane has been established as a part of the prerequisite information listed previously. The lack of an exclusive left turn lane does not necessarily preclude left turn protection, but it does constrain the available phasing plan choices to be discussed later in this chapter.

4.6.4 Protected vs. Protected-Permitted Left Turn Treatment

Protected-permitted operation will generally improve both capacity and driver convenience for left turning traffic. At the same time, however, it may introduce hazards because of the conflicts between the left turns and the opposing traffic. There are several examples in Florida in which protected-permitted left turn phasing was eliminated because of an unusually high crash experience.

Protected-only phasing should be used at all intersections where protected-permitted phasing is likely to present a hazard to left turning traffic or pedestrians that conflict with left turning traffic. There are specific conditions that explicitly preclude left turns on permitted phases as indicated in the box at the right. As a general guideline, protected only phasing may be preferred when:

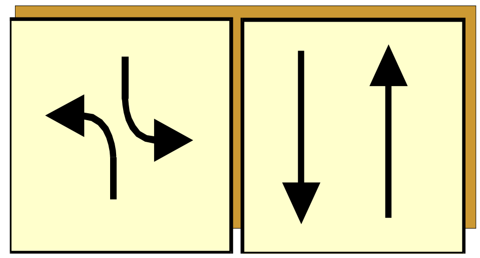
> **Conditions Requiring Protected-only Left Turn Treatment**
>
> Two or more left turn only lanes are provided.
>
> Geometric conditions and resulting sight distance necessitate protected only mode.
>
> The approach is the lead portion of a lead/lag intersection phasing sequence.
>
> The use of offset left turn lanes to a degree that the cone of vision requirements in Section 4D.13 of the ***MUTCD*** for the shared signal display cannot be met.
>
> The outside tracking paths of opposing left turning vehicles overlap.

1. The posted speed is greater than 45 mph,
2. The left turns must cross three or more lanes of through traffic,
3. There are more than six left turn crashes per year on an approach,
4. The intersection geometric configuration is unusual or complex.

It is also important to consider the impact of permitted left turn phases on the safety of pedestrian and bicycle traffic that could produce conflicts (sometimes unexpected) with left turning vehicles. Bicycles are especially vulnerable to these conflicts because of their higher speeds and the greater challenge to the reaction times of motorists who are expected to yield to them.

4.6.5 Leading vs. Lagging Left Turns

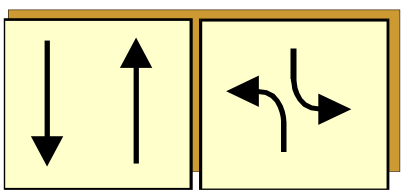
Protected left turns that precede the through movements are referred to as leading left turns. Protected left turns that follow the through movement are referred to as lagging left turns. The predominant practice in Florida strongly favors leading left turns. Lagging left turns may offer some benefit under the following conditions:



Leading left turn protection

1. Modeling of arterial operations indicates that better progression may be achieved by a lagging left turn in one direction accompanied by a leading left turn in the other direction. This scheme is generally referred to as "lead-lag" phasing.

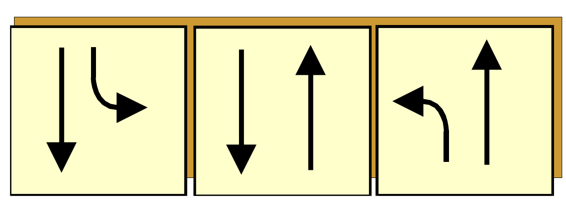
2. Unpredictable left turn peaks occur on approaches on which left turn volumes normally don't justify protection. It is a recognized practice to employ a permitted left turn phase followed by a lagging protected phase, with the left turn detector placed two or three car lengths behind the stop line. This will prevent the left turn arrow from being displayed unless a sufficient queue exists at the end of the permitted phase. This scheme offers a more efficient signal operation, but creates a potential for confusion among motorists who expect a green arrow display on every cycle.



Lagging left turn protection

3. High crash rates are observed between opposing left turns, often in the form of sideswipes. This hazard can be greatly reduced or eliminated by lead-lag phasing which accommodates the conflicting left turns at different times in the cycle.

It is important to note that, while protected-permitted left turn phasing and lead-lag phasing are both legitimate and recognized practices, the combination of these two schemes can produce a serious hazard, known as the "Left Turn Trap." This term denotes a condition in which a permitted left turn phase ends in one direction while the opposing through movement continues through the succeeding phase. A hazard is introduced under this condition because the left turning drivers tend to perceive the end of their phase as an opportunity to clear the intersection as a "sneaker," while the green indication in the opposing direction is displayed continuously during the transition from one phase to the next. Therefore the use of lagging left turns should be restricted to the following situations:



Lead-lag phasing

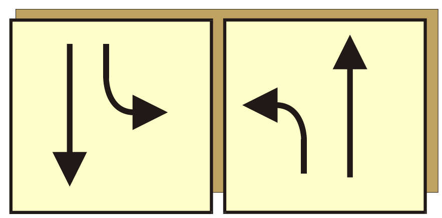
1. "T" intersections where opposing U turns are prohibited;
2. Four-way intersections where left turns are prohibited on the opposing approach or when the leading approach has protected only left turn phasing.

3. Four-way intersections where the left turn volumes from opposing approaches do not substantially differ throughout the various time periods of a normal day, so that overlapping phasing is not beneficial or required.

While the left turn trap is generally associated with lead-lag operation, it will occur on any phase transition in which a permitted left turn display in one direction is followed by a through/left display in the opposite direction. This will happen on each cycle regardless of whether the permitted phase follows a leading protected phase.

It must also be recognized that the left turn trap may arise whenever unusual phase transitions occur as a controller skips phases under low volume conditions. This is inherent in the operation of traffic-actuated controllers. Such transitions are a rare occurrence and are generally not accompanied by significant numbers of left turn sneakers. This phenomenon is beyond the control of the designer and the only way to circumvent the problem with multi-phase operation is to eliminate all permitted left turns.

4.6.6 Split-Phase Operation



Split-phase operation

The term "split-phase" is commonly used to describe a phasing scheme with full directional separation between the movements on opposing approaches, as illustrated in the box at the right. This type of operation is normally less efficient in terms of intersection capacity and performance than phasing schemes in which the opposing through movements proceed concurrently. For this reason, split-phase operation is normally restricted to situations in which:

1. Opposing approaches are offset such that simultaneous left turns could not be accommodated safely;
2. The volume distribution by movement would not reduce the efficiency of split-phase operation;
3. Left turns must be protected on an approach that does not include an exclusive left turn lane;
4. Left turns are made from both an exclusive and an optional lane; and,
5. Crash records indicate a high frequency of collisions between opposing left turns.

4.7 SELECTION OF RIGHT TURN TREATMENTS

Three parameters define the right turn treatment for each approach:

1. Lane utilization: shared, exclusive or channelized;
2. Right turn on red (RTOR): allowed or prohibited and
3. Signal protection: permitted, protected or both.

The lane utilization will usually be established with the development of the intersection configuration prior to the signalization design. It is, however, important to ensure that the lane utilization is compatible with the signal protection and with the accommodations for pedestrians. The decision to construct multiple right turning lanes requires more analysis and justification because of the increased crosswalk lengths and pedestrian conflict with right turns. The other two parameters are an essential part of the signal design and will be examined individually.

4.7.1 Need for RTOR

Florida Statutes permit RTOR unless specifically prohibited. Therefore, the default option is that RTOR will be permitted. Prohibition will generally be justified by safety considerations, brought about by high crash rates, visibility limitations, complicated geometrics or phasing and special populations. RTOR prohibitions may also be desirable on cross streets where heavy conflicting U-Turn volumes are observed. Note that RTOR may be prohibited at school signals as a matter of local judgment.

> "No right turn on Red" signs may be erected at school signals as deemed necessary by local traffic engineers.
> ***{Design Standards, Index 17344}***

4.7.2 Need for Protected Right-turn Phasing

The ***MUTCD*** presents a set of definitions for right turn treatments that are analogous to the definitions for left turn treatments. (See Section 4.5.1.) Right turns proceed as permitted movements at the vast majority of signalized approaches, yielding to conflicting pedestrians and bicycles.

> A steady green arrow shall be used only when corresponding movements are protected from all conflicts, including pedestrians and bicycles.
> ***{MUTCD}***

There are, however, cases in which protected (i.e., right turn arrow) phases are desirable. At intersections with very heavy pedestrian traffic and vehicular green times that exceed pedestrian crossing time requirements, it may be beneficial to display a right turn arrow during a portion of the vehicular green to provide adequate capacity for the right turn. This could also benefit the pedestrian traffic by accommodating more of the right turns under conflict-free conditions. The pedestrian benefits could be further increased by prohibiting the right turns during the portion of the "through" phase used by pedestrians. This scheme is most practical to implement when exclusive right turn lanes are available and when green times are relatively long. Longer green times can be created by increasing the cycle length. Even though longer cycle lengths generally produce higher delays for both vehicles and pedestrians, the safety benefits may justify the sacrifice in other performance measures.

Protected right turns are easier to implement at intersections with protected left turn phasing. Each right turn movement has a “shadowing” left turn movement that, when protected, facilitates the right turn. For example, the northbound right turn may proceed unopposed by vehicles under RTOR during a protected westbound left turn phase. Under these conditions, the right turn capacity may be increased further by displaying a right turn arrow concurrently with the protected left turn phase. This feature produces a more complex phasing and is usually implemented only when required for adequate right turn capacity. When protection is required for a specific right turn, it is advisable to prohibit U-turns for the shadowing left turn movement to avoid the display of simultaneous protected green arrows to conflicting movements. In any event, it is necessary to observe the ***MUTCD*** requirement that green arrows be shown only to fully protected movements, including protection from bicycles and pedestrians. This requirement could be satisfied by installing a “U Turn Yield to Right Turn” (R10-16) sign facing the direction from which the U turn originates.

4.8 DETERMINATION OF THE REQUIRED LENGTHS FOR TURN LANES

The 2010 version of the Highway Capacity Manual provides means to estimate the back-of-queue length for selected lane assignments and grouping. The back-of-queue is defined as the point reached just before the most distant queued vehicle begins forward motion as a consequence of the green indication and in response to the forward movement of the vehicle ahead. The queue length includes only fully stopped vehicles; vehicles that slow as they approach the back-of-queue are considered to incur a *partial stop* but are not considered part of the queue. The back-of-queue size represents the average queue for the analysis period (usually 15 minutes), and is based only on those vehicles that arrive during the analysis period and join the queue. It includes the vehicles that are still in the queue at the end of the analysis period.

The back-of-queue is computed for specific lane groups, and is expressed in vehicles per lane. The length of storage provided for that lane group should equal or exceed the back-of-queue distance, assuming a vehicle length of 25 feet. Perception-decision-reaction and maneuver distances (see **Figure 2-3**) should be calculated based on Stopping Sight Distances as described in the ***AASHTO Green Book***.

The ***2010 HCM*** has been updated to include a new procedure for estimating queue lengths, termed the Queue Accumulation Polygon (QAP). The QAP is a deterministic model describing the relationship between vehicle arrivals, departures, queue service times, and delay. The QAP defines the queue size as a function of time during the cycle, and is best calculated using computer software.

4.9 SELECTION OF CONTROL EQUIPMENT

> Traffic signal equipment installed in Florida is required to be certified by the FDOT's Central Traffic Engineering Office. Non-certified equipment cannot be used.

Department certification is required for each item of control equipment installed. Certification procedures are summarized in the ***TEM***. The product of the certification process is the Approved Products List (APL). To receive certification, a product must meet all requirements of the ***MUTCD***, FDOT ***Minimum Specifications for Traffic Control Signal Devices (MSTCSD)***, and FDOT ***Standard Specifications for Road and Bridge Construction****.*

The review process involves an initial review based on documentation submitted by an applicant. The FDOT requires a subsequent review of the actual product, performed on hardware sample(s) furnished by the applicant. Products that are determined to meet the required criteria receive a certification number and are added to the APL. The Department conducts periodic follow-up reviews of certified products to ensure that conformance to requirements is maintained. The APL is available on the Traffic Engineering and Operations website at:

http://www3.dot.state.fl.us/trafficcontrolproducts/.

In addition, Institute of Transportation Engineers (ITE) offers published standards for the following items of intersection control equipment:

1. Pre-timed Traffic Signal Controllers;
2. Traffic-Actuated Traffic Signal Controllers: solid state;
3. Solid-State Pre-timed Traffic Signal Controller Units;
4. Controller Cabinets;
5. Traffic Signal Lamps;
6. Vehicle Traffic Control Signal Heads;
7. Lane-Use Traffic Control Signal Heads;
8. Pedestrian Traffic Control Signal Indications;
9. Purchase Specification for Flashing and Steady Burn Warning Lights; and,
10. A Standard for Vehicle Detectors.

The majority of signalized intersections now employ dual-ring traffic actuated controllers conforming to NEMA standard TS-2. Inductive loop detectors are one of the most common choices for vehicle detection, although video detection may soon overtake loops across the state. The control equipment to be installed at coordinated intersections must be compatible with the master control equipment. In many cases, this will dictate a vendor-specific product choice. Designers should check with the local maintaining agency for preferences.

4.10 SELECTION OF THE PHASING PLAN

The signal sequence is controlled by a group of interconnected phase modules, each of which generates the commands to control a single green, yellow, red, walk and don't walk display, with proper transitions between displays. Each module accepts vehicle detector inputs that indicate whether or not a vehicle is present on the approach controlled by the phase. Only one detector input is recognized per phase.

Vehicular presence during the red interval will place a demand for a green signal to be displayed. The detector is ignored during the display of the minimum green interval, but after this interval has expired, the presence of a vehicle will cause the phase to be extended by a fixed duration (called "extension time") until a gap of the prescribed length has been encountered.

Each phase module also has a single input for pedestrian demand. If vehicular demand is received without pedestrian demand, the phase timing will be as described above. In such case the pedestrian signal will display a constant "don't walk" indication. If pedestrian demand is received, with or without vehicular demand, the phase module will display a "walk" interval of prescribed duration followed by a "don't walk" interval. After the pedestrian timing has been satisfied, the normal phase extension process will be invoked until a gap in the traffic causes the phase to terminate.

The signal controller consists of a group of these phase modules interconnected to pass control from one to another in an orderly fashion. Two examples of the representation of signal phasing, as presented in the ***Design Standards***, ***Index 17870***, are shown in Figure 4-3 and Figure 4-4. In the simplest form of operation, only one phase module will assume control at any time. The rest will remain in the red state, but will register a demand for their turn whenever they detect vehicular or pedestrian presence.

This simple sequence is known as *single-ring sequential* operation. In its most elementary form, only two phase modules will be involved in the sequence, with each controlling an opposing pair of approaches. Additional modules may be added to a single ring operation to provide exclusive phases for left turns or other minor movements. An example of single ring operation is presented in ***Figure 4-3***, with four phases displayed sequentially as the terminology suggests. Note that each phase module controls two approaches that move simultaneously.

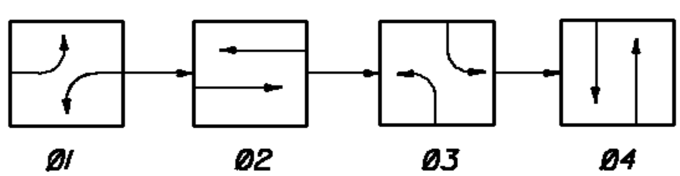


Figure 4-3 Single Ring Signal Sequential Operating Plan

Most traffic-actuated intersections in Florida use a more complex scheme known as *dual-ring concurrent* operation, which is depicted in ***Figure 4-4***. Note that Phases 2 and 5 in the phasing diagram offer two alternatives, only one of which may be displayed on any cycle.

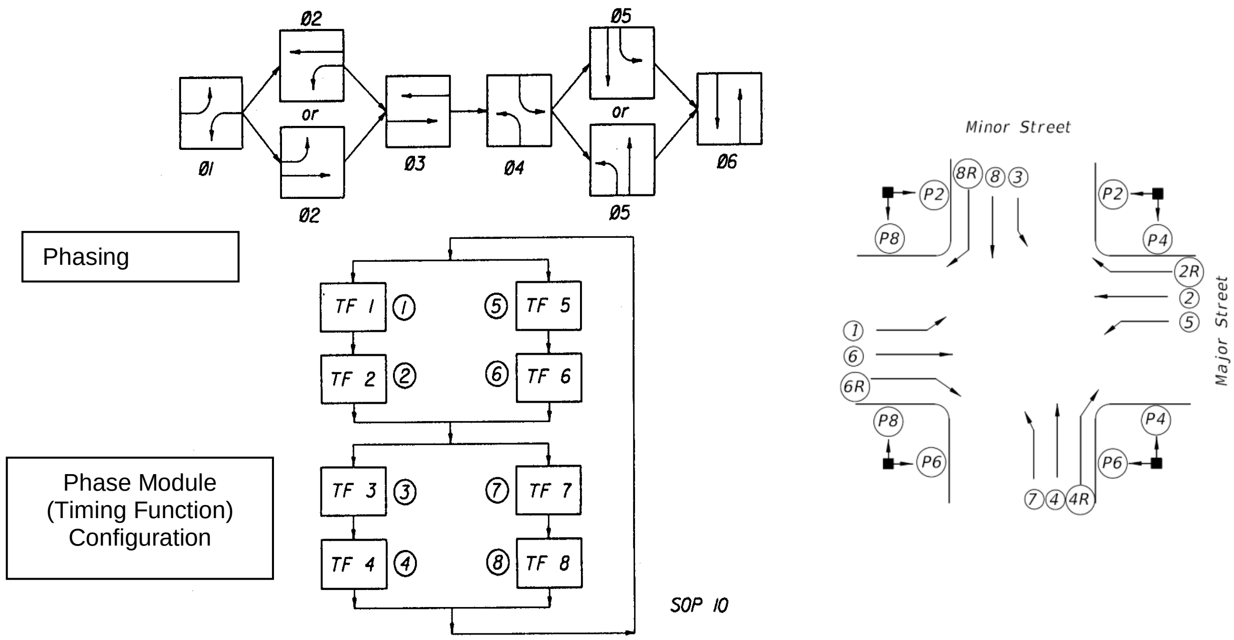


Figure 4-4 Representation of Dual-Ring Concurrent Signal Phasing

By NEMA convention, timing functions (TF) 1 through 4 constitute Ring 1 and TF5 through 8 constitute Ring 2. A functional barrier exists between TF 2/6 and TF 3/7. The control in both rings must cross the barrier at the same time to avoid displaying a green indication to two conflicting movements.

Dual-ring concurrent phasing is generally more efficient than single-ring sequential phasing because it allows the available green time to be distributed with less "slack," i.e., unused green time. ***Design Standards, Index 17870*** refers to the phase modules as *Timing Functions* (TF). TF 1-4 constitutes Ring 1 and TF 5-8 constitutes Ring 2. Each timing function controls only one movement. Since two movements can proceed simultaneously without conflict, two of the timing functions (one from each ring) will always have control simultaneously. This process is transparent to the road user. The phasing plan selection should be based on one of the standard signal operating plans prescribed in ***Design Standards, Index 17870*.**

4.11 SIGNAL TIMING

Signal timing involves the determination of the appropriate cycle length (i.e., the time required to execute a complete sequence of phases) and apportionment of time among competing movements and phases. The timing apportionment is constrained by minimum green times that must be imposed to provide for pedestrians and to ensure that motorist expectancy is not violated.

The Florida Board of Professional Engineers requires the signature and seal of the Engineer of Record on Signal Timing Plans

Within these constraints, there are three objectives that may be chosen to optimize the timing design:

a. Equalization of the degree of saturation (volume/capacity ratio) among competing movements: This design is usually associated with simple manual methods of signal timing because it is the only approach that can be implemented effectively without computer software. It is also the starting point for many software-based procedures.

b. Minimization of the aggregate delay to all vehicles using the intersection: This is the only objective that could be considered as a true optimization. The others are based on equalizing driver-perceived disutility. Some procedures consider stops, fuel consumption and operating cost in addition to delay in the optimization process.

c. Equalization of control delay among competing movements: While this is not a true optimization, it is popular because the ***HCM*** defines level of service in terms of delay to each vehicle. Therefore the equalization of delay results in a more equitable assignment of intersection level of service among all road users.

The software products used for designing signal timing plans are identified in Chapter 4 and the specific parameters to be determined by the signal timing process are discussed.

4.11.1 Assignment of Movements to Timing Functions

The complete specification of the phasing plan requires that each of the traffic movements to be accommodated be assigned to one of the eight timing functions to produce the desired sequence of displays. The choice of left turn treatments discussed previously will determine which timing functions will be activated and which will be omitted from the phasing plan. One-way streets and “T” intersections can also cause certain movements and timing functions to be omitted. When the set of active phases has been determined, it is necessary to assign each of the traffic movements to a specific timing function.

4.11.2 NEMA Convention

The National Electrical Manufacturers Association (NEMA) establishes standards for nomenclature, operating characteristics, performance, etc. of manufactured electrical and electronic devices, notably including traffic signal controllers. NEMA "convention" offers some latitude in the assignment of traffic movements to timing functions. The following guidelines are offered for traffic movement assignment:

1. *Protected Left Turns:* It is common to assign the left turn movements to odd numbered timing functions because the left turns generally precede the through movements (even numbered timing functions) on any given approach.
2. *Arterial Movements:* It is common to assign the arterial movements to the left side of the barrier (TF 1, 2, 5 and 6). Using this convention, TF 2 and 6 will be coordinated by the master controller and the remainder of the timing functions will be controlled by detectors.
3. *Ring Assignments:* On both sides of the barrier, either left turn can be assigned to Ring 1, with the opposing left turn assigned to Ring 2. This choice will determine the assignment of the through movements to the other timing functions. This choice is more or less arbitrary. Many agencies have adopted the convention used by PASSER II, a signal timing design program to be described later. This convention depends on the choice of arterial orientation. The PASSER movement numbering convention is presented in ***Figure 4-5***. The use of this convention will facilitate the transfer of timing parameters from the timing design program to the field hardware.

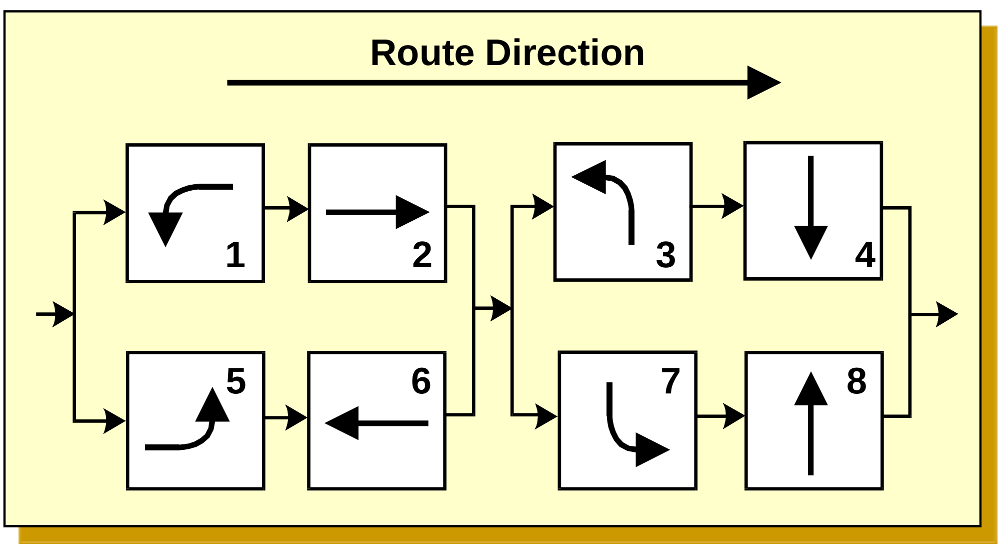


Figure 4-5 PASSER II Movement Assignement Convetions

4. *Permitted Left Turns:* When left turns are permitted, no green arrow is displayed and therefore the movement is not assigned to a timing function. The left turning traffic is accommodated on the solid green displayed by the timing function for

through traffic. When two opposing left turns are omitted, it is possible to combine the through movements to be controlled by a single timing function, because both movements are displayed at exactly the same time. While this practice may simplify the field equipment configuration, it could degrade the performance of the intersection. Each through movement should therefore be assigned to its own timing function, even in the absence of protected left turns.

5. *Lagging Left Turns:* Lagging left turns follow the through movement in any phase pair. There are two ways to implement lagging left turns.
    a. The left turn may be assigned to the even numbered timing function with the through movement assigned to the odd number. The assignment is accomplished by the controller cabinet wiring.
    b. Most controllers provide a "phase reverse" feature that is not strictly a NEMA standard. When activated, this feature instructs the controller to display the timing functions in reverse order.

It is common practice to use the latter alternative to preserve the movement numbering convention of ***Design Standards, Index 17870*** on controllers that offer this alternative.

4.11.3 Selecting the Standard Signal Operating Plan

The standard Signal Operating Plan (SOP) is normally used in place of a phasing diagram in the preparation of signalization plans for FDOT projects. The SOP is characterized by the choice of phase plans for the north-south and east-west movements, respectively. ***Table 4-2*** shows a matrix of the SOP numbers that have been assigned in ***Design Standards, Index 17870*** to the various movement combinations. For example, SOP 6 is characterized by leading-protected left turns in one direction and split-phase operation in the other direction.

> If the phasing plan conforms to an SOP shown in Design Standards, Index 17870, then the index reference may be used in place of the phasing diagram.

Note also that this index presents SOP's for "T" intersections and one-way streets (SOP 12 and 13, respectively) that are not represented on ***Table 4-1***. SOP's have also been developed for diamond interchanges (SOP 14, 15, 18 and 19) and mid-block pedestrian signals (SOP 17). In addition, three preemption operating plans (POP's) are presented for application at railroad grade crossings.

In some cases no SOP number has been assigned, as indicated by blank cells in ***Table 4-1***. Phasing diagrams must therefore be drawn on the signalization plans for these cases. There are, in addition, a few specific situations where the phasing is too complex to represent by a standard operating plan. Examples include:

1. Exclusive Pedestrian Phases: Where pedestrians receive an exclusive phase free of all vehicle movements, special phasing must be used. If all of the left turns have protected phases, a special controller with more than eight phases is required. If

an operating plan is used that includes less than 8 phases, then there will be a timing function available for assignment to an exclusive pedestrian phase.

2. Protected Pedestrian Phases: These will not usually affect the assignment of through movements, but they could influence the assignment of left and right turns. When sufficient capacity exists, traffic signal heads that restrict left and right turns during the pedestrian phase will provide full protection from conflicting vehicular movements.

3. Concurrent Pedestrian Phases: Pedestrian crossings can be facilitated concurrently with the corresponding through movement, but the timing function may have a greater minimum green time to accommodate the longer crossing time of pedestrians. Conflicting right or left turns are legally required to yield to pedestrians within the crosswalk during the concurrent pedestrian phase.

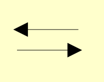
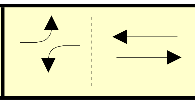
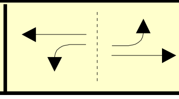
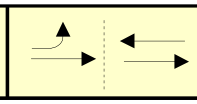
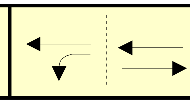
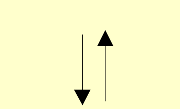
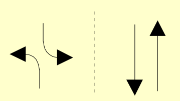
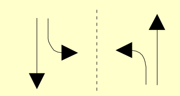
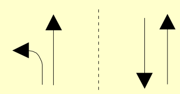
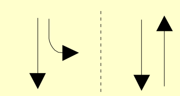
DEVELOPMENT OF THE TIMING PLAN(S)

The development of signal timing plans is a process that depends heavily on reliable data and appropriate software combined with the professional experience of the designer. The Board of Professional Engineers requires the signature and seal of a professional engineer on all signal timing plans. While the process involves many quantitative computations, it also requires a creative approach to the identification of feasible alternatives. A substantial body of knowledge beyond the scope of this document must be called upon. The process can only be summarized here. A general knowledge of the operating principles and characteristics of a traffic-actuated controller is assumed in this discussion.

> All traffic signal designs prepared for or by the Department shall include initial timings of all controllers.
>
> ***{PPM}***

> Since traffic flows and patterns change, phasing and timing should be re-evaluated regularly and updated if needed.
>
> ***{MUTCD}***

Table 4-1 Standard Signal Operating Plan Numbers Assigned to Various Combinations of Movements in the Phase Plan

						Full Dual Ring
	1	2	3	11	11	7
	2	4	6			8
	3	6	5			9
	11			16	16	
	11			16	16	
Full Dual Ring	7	8	9			10

4.11.4 Information Requirements

Several signal control choices or decisions that precede the detailed timing plan development have already been discussed, including:

1. Left turn treatments;
2. Right turn treatments;
3. Phasing plan, including the assignment of movements to timing functions; and,
4. Signal Operating Plan selection.

Other site specific information that will influence the timing plan computations includes:

1. Hourly vehicular and pedestrian demand volumes for all movements;

2. Factors that will affect the capacity of each approach, including grades, parking, lane widths, traffic composition, turning proportions, any unusual site characteristics, and;
3. Constraints imposed by the system, especially the cycle length requirements of other intersections.

> On bikeways, signal timing and actuation shall be reviewed and adjusted to serve the needs of bicyclists.
>
> ***{MUTCD}***

4.11.5 Minimum Phase Times

One very important determinant of phase times is the minimum duration for which a phase may be displayed. Minimum phase times must be imposed as a constraint on the design when the theoretical optimum phase duration (e.g., that which would minimize vehicular delay) suggests a lower value. All of the practical signal timing design software products to be discussed later recognize minimum phase time constraints.

> Where pedestrian movements regularly occur, pedestrians should be provided with sufficient time to cross the roadway by adjusting the operation and timing to provide sufficient crossing time every cycle or by providing pedestrian detectors.
>
> ***{MUTCD}***

The two principal controls that govern minimum phase times are driver expectancy and pedestrian crossing time requirements. Driver expectancy dictates that a green signal duration should not be so short that the driver reacts improperly (e.g., panic stop or violation of the red), based on his or her experience. Driver expectancy will determine minimum phase durations for phases that do not need to accommodate pedestrians, including:

1. Protected left turn phases;
2. All phases at traffic-actuated controllers in which a separate pedestrian timing is generated in response to pedestrian demand; and,
3. Phases controlling approaches on which pedestrian movements have been prohibited.

Commonly accepted values for minimum green display times based on driver expectancy are 8 seconds for protected left turns, 12 seconds for through movements on minor roads and 20 seconds for through movements on major arterial roads. Note that the intergreen interval (yellow change plus all red clearance) must be added to the minimum green times to determine the minimum phase durations.

Pedestrian crossing time requirements will generally dictate the minimum phase times on all phases that must accommodate pedestrians. The pedestrian crossing time requirement is determined as the sum of two components, including the startup time and the clearance time. The startup time corresponds to the steady green walk interval and should be at least 7 seconds in length so that pedestrians will have adequate opportunity to leave the curb or shoulder before the pedestrian clearance interval time begins. If pedestrian volumes and characteristics do not require a 7-second walk interval, walk intervals as short as 4 seconds may be used. The clearance time should be determined as L/S_p , where L is the crosswalk length and S_p is the assumed walking speed of pedestrians. The commonly accepted value of S_p is 3.5 ft./sec. This value should be used unless site-specific conditions suggest otherwise. For example, if a significant number of elderly pedestrians routinely cross at an intersection, a walking speed of less than 3.5 ft/s should be considered. The pedestrian clearance time will determine the length of the flashing don't walk and countdown pedestrian signal indications.

4.11.6 Timing Plan Parameters

There are two levels of timing plan parameters that must be determined. The first is the intersection level. For a traffic-actuated controller, the most important parameters to be determined for each controller timing function to be activated include:

1. The unit extension time or size of the detected gap in the traffic that will cause the phase to terminate. The controller assumes that the queue of traffic accumulated on the red phase has been serviced when a gap beyond this threshold is observed. An approximate but rational approach would be to double the average gap that would be expected between vehicles departing from the queue based on the saturation flow rate and the number of lanes.
2. The initial interval time that will be displayed before the gap searching process begins: This interval determines the minimum green time for the timing function. It must be long enough to start the queue moving smoothly across the detector and to ensure that the previously established minimum green time is displayed.
3. The maximum duration that the timing function can retain control (i.e., display a green signal). The maximum green time is an essential override that prevents a movement from retaining control for an unreasonable length of time. This time should never be shorter than the optimal green time determined by the timing design and analysis software for the heaviest peak period. It is common practice to add 10 to 20 percent to the optimal green time to give the controller the flexibility to accommodate variations in traffic demand from cycle to cycle. Traffic-actuated controllers typically provide two maximum interval settings that may be chosen externally, usually based on time of day.
4. The intergreen timing parameters (yellow and all-red), determined as indicated in Sections 4.11.8 and 4.11.9. The common software products used for timing design and performance evaluation do not determine the intergreen parameters. These computations must be performed externally.
5. The WALK and flashing DON'T WALK intervals of pedestrian displays are controlled by the timing function. The minimum length of the flashing DON'T WALK interval should be based on the assumed pedestrian crossing speed. The minimum length of the WALK interval is subject to local preference within constraints imposed by the ***MUTCD***. These two pedestrian intervals

> The WALK interval should be at least 4 to 7 seconds in length. A pedestrian clearance interval shall always be provided where pedestrian signal indications are used.
>
> ***{MUTCD}***

override the minimum green time established by the initial interval on cycles that receive pedestrian actuations. On cycles with heavy vehicular traffic, the maximum green time can extend the phase time beyond the pedestrian timing requirement.

6. The "recall" status of the timing function specifies the action to be taken in the absence of vehicle detection. The options are not to display the green at all, to display it for its minimum or maximum time or to invoke the full timing for the pedestrian control signals. It is common practice at isolated signals to recall only the timing function controlling the heaviest movements to encourage the right-of-way to remain in that position during periods of no demand.

These parameters represent the minimum specification for each timing function at an isolated signal. There are other more advanced parameters beyond the scope of this discussion that may be used to refine the operation.

For intersections that are coordinated with other signals along a corridor, a second set of timing parameters must be specified at the system level. The system parameters include:

1. The cycle length, which must be common to all intersections within the system. Arterial cycle lengths typically range from 60 to 150 seconds. The longer cycle lengths are generally applied during the peak periods.
2. The timing functions to be designated for coordinated movements. Under normal conditions TF2 and TF6 will be so designated because they handle the through movements on the arterial street. The remainder of the timing functions will operate in a traffic-actuated mode with detector inputs. The coordinated timing functions do not recognize detector inputs. All of the time in any given cycle not used by the actuated movements will revert to the coordinated movements. This has the desirable effect of giving more time to the arterial movements during periods of light cross street traffic.
3. Nominal phase times for each movement. These times must be translated into specified times for each timing function such that their sum adds up to the system cycle length. The actual green times for traffic actuated phases may be shorter than the nominal phase times if the traffic demand on any given cycle is not able to sustain the green time for its full duration. The specified times will override the maximum green times on each of the timing functions operating in the traffic-actuated mode.
4. An "offset" time that establishes when a specific point in the sequence will occur with respect to a common system reference time. The offset determines the time at which the arterial green is displayed at each intersection relative to its neighbors, thereby controlling the progressive movement of traffic. The offset is applied to different points in the sequence by different vendors.

4.11.7 Signal Timing Software Products

System level parameters are generally determined using software products that model the operation of traffic to optimize performance in terms of such measures of effectiveness as stops, delay, and fuel consumption, or progression quality as perceived by the motorist. Multiple software products exist and are in common use.

4.11.8 Yellow Change Interval Requirements

A yellow change interval is always required to provide a safe transition between two conflicting traffic signal phases. The function of a yellow change interval is to warn traffic of an impending change in the right of way assignment. ***Figure 4-6*** shows the clearance table presented in ***Design Standards, Index 17870***. This table indicates the requirement for and the nature of the clearance display that must follow each display of a solid green or green arrow.

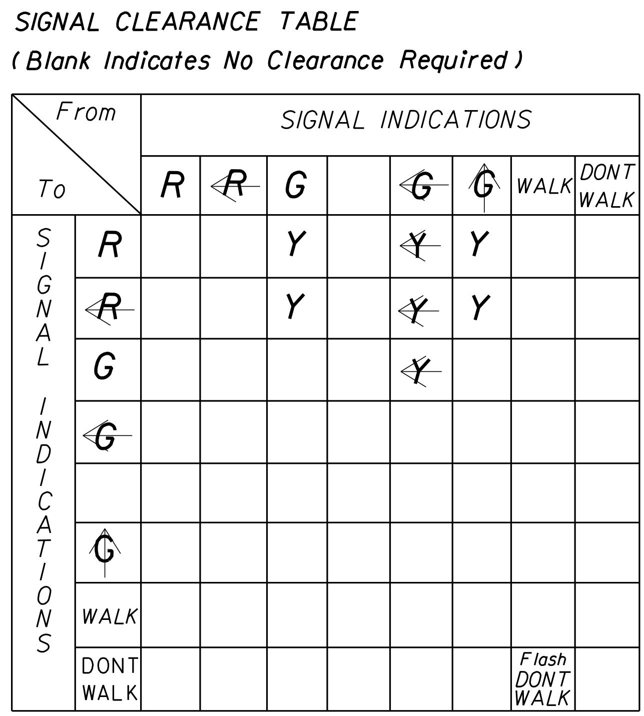


Figure 4-6 Signal Clearance Table from Design Standards, Index 17870

The yellow change interval must be long enough to avoid creating a “dilemma zone” which is defined as a condition under which the motorist finds it difficult to decide whether to stop or proceed through the intersection. The MUTCD states that a yellow change interval should have a minimum duration of 3 seconds and a maximum duration of 6 seconds. A widely used equation for determining the minimum length of yellow change interval to avoid the dilemma zone is:

$$Y = t + \frac{1.47v}{2(a+Gg)}$$

where:

Y = length of yellow interval, sec.
t = perception-reaction time, (Use 1 sec.)
v = approach speed, mph
a = deceleration rate (Use 10 ft/sec^2)
G = grade, with uphill positive and downhill negative (% grade/100)
g = acceleration due to gravity (32.2 ft/sec^2)

This equation is found in the ITE ***Traffic Engineering Handbook***. A yellow change interval shorter than this will create a potential situation in which the driver may not be able to stop before reaching the intersection or enter the intersection before the end of the yellow as required by ***Florida Statutes***.

4.11.9 All-Red Clearance Interval Requirements

The yellow change interval must be followed by an all red clearance interval of sufficient length to allow a vehicle that has entered the intersection at the end of the yellow change interval to clear that intersection before the onset of the green display for any conflicting movements. This interval length may be computed as:

$$R = \frac{W+L}{1.47v}$$

where:

R = all red Clearance interval, sec
W = width of intersection from stop bar to far side of no-conflict point, ft.
L = length of vehicle, (Use 20 ft.)
v = approach speed, mph

A clearance interval shorter than the one computed above could cause vehicles to be trapped within the intersection after the beginning of a green display for conflicting traffic movements.

While the computations for the yellow change and all red clearance intervals are mathematically independent, it is common practice to limit the yellow interval display to 5 seconds and to assign any additional time to the all red interval, ensuring that the sum of the two intervals meets the minimum requirement.

4.11.10 Pedestrian Walk and Clearance Time Requirements

When pedestrian signal heads are used, a Walk signal indication shall be displayed only when pedestrians are permitted to leave the curb or shoulder. A pedestrian clearance time shall begin immediately following the Walk signal indication. The first portion of the pedestrian clearance time shall consist of a pedestrian change interval. The remaining portions shall consist of the yellow change interval and any red clearance interval. During the transition into preemption, the walk interval and the pedestrian change interval may be shortened or omitted.

At intersections equipped with pedestrian signal heads, the pedestrian signal indications shall be displayed except when the vehicular traffic control signal is being operated in the flashing mode. At those times, the pedestrian signal lenses shall not be illuminated.

The walk interval should be at least 7 seconds in length so that pedestrians will have adequate opportunity to leave the curb or shoulder before the pedestrian clearance time begins. If it is desired to favor the length of an opposing signal phase and if pedestrian volumes and characteristics do not require a 7-second walk interval, walk intervals as short as 4 seconds may be used.

The pedestrian clearance time should be sufficient to allow a pedestrian who left the curb or shoulder during the Walk signal indication to travel at a normal walking speed of 3.5 feet per second, to at least the center of the farthest traveled lane or to a median of sufficient width for pedestrians to wait. Where pedestrians who walk slower than normal or pedestrians who use wheelchairs, routinely use the crosswalk, a lower walking speed should be considered in determining the pedestrian clearance time, as discussed in Section 4.11.2. Passive pedestrian detection equipment, which can detect pedestrians who need more time to complete their crossing and can extend the length of the pedestrian clearance time for that particular cycle, may be used to avoid using a lower walking speed to determine the pedestrian clearance time.

4.11.11 Interconnection Requirements

> Signals within ½ mile on major routes should be coordinated.
>
> ***{MUTCD}***

Signalized intersections may operate in an isolated mode, independent from all other intersections or they may be a part of a traffic control system. In addition, their proximity to other influences such as railroad grade crossings, drawbridges or fire stations might require interconnection for preemption purposes. Isolated intersections will nearly always use standard eight phase dual ring controllers of the type described earlier in this chapter. Interconnected systems tend to have vendor-specific features that could influence the timing design.

> Signals within 200 ft of a railroad grade crossing should be interconnected for preemption by the railroad crossing protection.
>
> ***{MUTCD}***

Intersection spacing is the most important consideration in the decision to interconnect signals for progressive movement between intersections. It is commonly accepted that signals within ½ mile will always benefit from coordination. On well-designed arterial routes with minimal cross street entry and smooth traffic flow, coherent platoons of vehicles will remain intact for much longer distances.

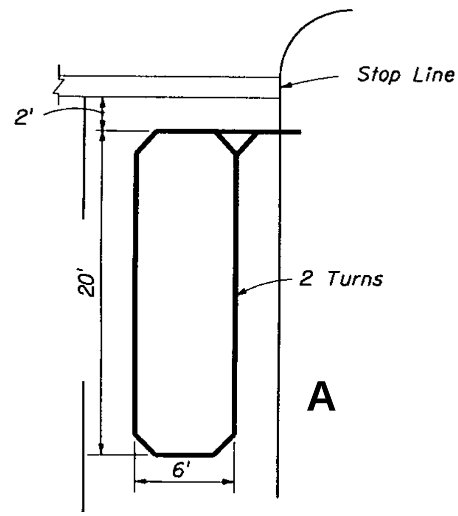
4.12 DETERMINATION OF THE DETECTOR CONFIGURATION

The detectors at a signalized intersection serve the singular purpose of informing the controller that a vehicle is (or is not) present on a particular approach to an intersection at any point in time. The controller uses this information to regulate the assignment of green time among competing movements at the intersection. The detector configuration is specified in terms of type, shape, length and setback from the stop line. These four parameters are all related to the physical installation of the detectors themselves, although they can have a direct impact on the signal operation.

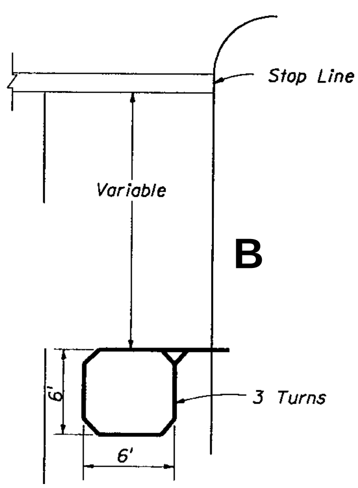
Vehicle detectors require FDOT traffic control device certification. There are several types of detectors on the market, each of which exploits a different principle of physics to detect the presence of a vehicle. The inductive loop detector has been the predominant detector type for more than 30 years because it has demonstrated the ability to detect vehicles reliably at a lower cost than competing technologies. The "loop" consists of a few turns of wire embedded in the roadway surface. The metallic presence of a vehicle on the loop is detected electronically by a unit in the controller cabinet.

4.12.1 Loop Detector Types

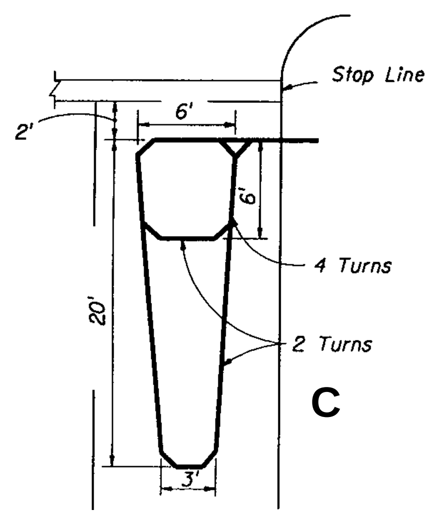
The size, shape and location of the loop establish the detection area*.* ***Design Standards, Index 17781*** prescribes seven detector shapes, labeled Type A through Type G. Each shape is suited to a particular purpose and some regional preferences exist for different types. Cut-off corners provide for ease of installation and prevent "kinking" and possible failure of the wire. The detector types are summarized as follows:



Type A Loops: consist of a simple rectangular shape generally located at or close to the stop line. The standard dimensions are 6 feet by 20 feet. This is the simplest type of loop to install. Its main drawback is that the area of influence tends to extend well beyond its lateral boundaries, creating a "spillover" effect that could cause false detection in adjacent lanes. It is also the least sensitive to bicycles. On the other hand, this shape is most sensitive to high-bodied vehicles such as semi-trailers.



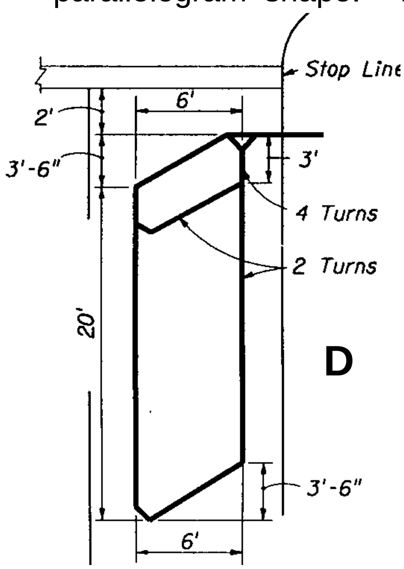
Type B Loops: are very similar to Type A except that they are square and smaller in size. The standard dimensions are 6 feet by 6 feet. They do not generally cover sufficient area to detect all vehicles at the stop line because of the wide range of stopping positions. Type B loops are best suited to counting traffic. They are used most commonly as system level sensors in arterial control systems. They are also placed some distance from the stop line at isolated rural intersections where speeds are higher and the control strategy is oriented more toward minimizing stops than maximizing capacity. A smaller version is also used in bicycle lanes and paths



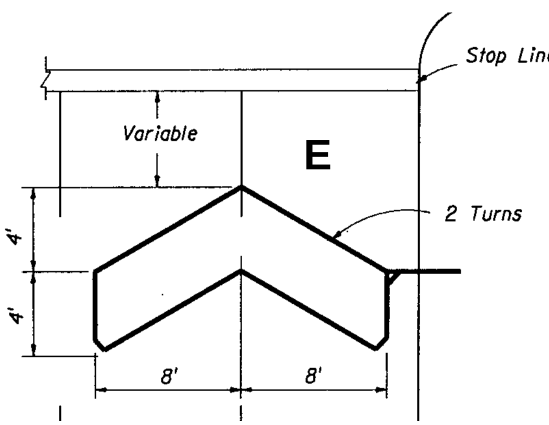
that are under signal control and where the bicycle positions are constrained so as to be predictable.

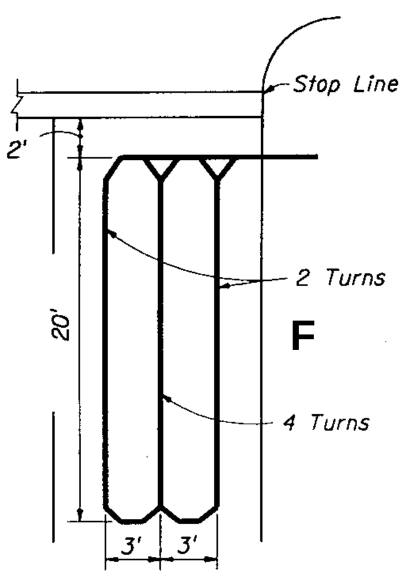
Type C Loops: have the same general dimensions as Type A loops, except that they are trapezoidal in shape, instead of rectangular. They also have a supplementary loop consisting of a few turns at the downstream end to form a “booster” which increases sensitivity to bicycles. The trapezoidal shape results in an angular vehicle motion over the loop wires, which increases the sensitivity to bicycles.

Type D Loops: have the same general dimensions as Type A and Type C loops, in a parallelogram shape. This shape implements the same principle of introducing an angular motion of the vehicle with respect to the loop wires to increase the sensitivity to bicycles and small vehicles.



Type E Loops: have a chevron shape that carries the concept of bicycle detection one step further. In addition to the angular motion, the chevron reduces the longitudinal distance between the loop wires to strengthen the sensitivity at levels closer to the pavement. This is not a good choice for detecting high-bodied trucks or vehicles that could stop in a wide range of positions at the stop line. It does, however, offer the ability to detect most moving bicycles anywhere within its boundaries. It is therefore a feasible alternative for application on wide shared use paths and at locations where the main objective is an accurate bicycle count. Type E loops are typically centered on two lanes.





Type F Loops: have gained a widespread acceptance for their sensitivity to bicycles and their minimal spillover into adjacent lanes. This “quadrupole” shape features three longitudinal sets of wires wound in a “figure 8” so that the center set of wires has twice as many conductors as the outside sets and carries current in the opposite direction with respect to traffic flow. The effect of this shape is to concentrate the sensitivity within the loop boundaries and to increase the sensitivity close to the ground. This compact zone of influence facilitates bicycle detection and prevents false detection in adjacent lanes. It also reduces the sensitivity to high-bodied trucks.

Design Standards, ***Index 17781*** also defines a Type G Loop, not shown here, which consists of a series of Type B (square) loops placed sequentially in the lane. A maximum of four loops may be used. This configuration provides a long detection zone without excessive spillover and places less strain on the loop conductors resulting from thermal expansion and contraction. It requires more effort to install and has lower sensitivity to bicycles than the quadrupole loop, but it has greater sensitivity to high-bodied trucks.

Some variation in the loop configuration details presented above is allowed on ***Design Standards***, ***Index 17781*** as shown in the box at the right.

> The maximum overall length of loop types A, C, D, and F is 60 feet, and their leading edge may extend up to 10 feet past the stop line.
> ***{Design Standards, Index 17781}***

4.12.2 Detector Location Guidelines

The following design guidelines are offered with respect to detector placement:

1. Vehicle detectors must be located where vehicles stop for the red signal otherwise demand for service will not be recognized.
2. Loops should be designed for installation before application of the final surface course where asphalt is being placed.
3. Loops are not necessary on main street dedicated, channelized right turn lanes.
4. Loops should not generally extend beyond the stop bar unless the intersection layout is such that vehicles have the space to store beyond the stop bar, or at existing locations where such behavior is documented.
5. Advance loops on the major street approaches generally produce the greatest benefits at the first signal in a coordinated system. As a matter of judgment, they may be installed on any major street approach on which the speed limit is greater than 35 mph.
6. Major street stop bar loops are not required in applications where advance loops are utilized unless driveways exist between the advanced loop and the stop bar. The minimum green interval should be set to service any queue that may build in the area between the stop bar and the first set of advance loops.
7. It should be determined if any system sensor loops are present or necessary at the intersection and if the construction activity will affect them. Project limits should be expanded to include any such loops and provisions for their replacement should be made as applicable.
8. Delay detectors should generally be used for side street right turn movements and for all permitted left turn movements. Delay for permitted left turn loops is especially important when the intersection layout is such that left turning vehicles from other approaches may impinge on a specific left turn lane by “corner cutting.”

9. Vehicle detection on the major street (i.e., coordinated phase) is not recognized when the intersection is operating as a part of a coordinated system. Detectors should, however, be installed on all approaches to provide for periods of uncoordinated operation.

4.12.3 Other Detection Technology

Above ground detection devices are preferred by a growing number of maintaining agencies, who should be consulted for guidance. Among the types of devices available are microwave based (which are used primarily in temporary applications), wireless magnetometer, and Video Image Detection (VID). Among the advantages of these types of devices are situations that preclude the installation of a loop in the roadway (e.g., unpaved roads, bridge decks, private driveways, etc.). One VID camera can cover several lanes of traffic, or an entire intersection when fitted with a “fisheye lens”.

4.12.4 Pedestrian Detectors

Pedestrian detection is accomplished by manual push buttons that connect directly to the controller timing function that controls the pedestrian signal display. When pedestrian control signals are provided, the activation of a pedestrian detector will invoke the WALK – DON’T WALK sequence to override the minimum green time for the vehicular display and to guarantee that the specified minimum crossing time for pedestrians is displayed. At locations without pedestrian signals, the pedestrian detector is connected to the controller in the same manner as the vehicle detector to ensure that a pedestrian will receive a green indication in the absence of vehicular demand. Phase recall should not be used as an alternative to providing pedestrian detection. When pedestrian signals are not installed, it is essential that the minimum green times for the vehicular movements satisfy the minimum time for pedestrian crossing discussed previously in this chapter.

Pedestrian push buttons should be located where they are conspicuous and convenient for both pedestrians and cyclists. ***Design Standards, Index 17784*** sets forth the requirements for the installation of pedestrian detector assembles, including the push button detectors and advisory signs. The advisory sign requirements are discussed in Chapter 5.

If two crosswalks, oriented in different directions, end at or near the same location, the positioning of pedestrian detectors and/or the legends on the pedestrian detector signs should clearly indicate which crosswalk signal is actuated by each pedestrian detector.

If the pedestrian clearance time is sufficient only to cross from the curb or shoulder to a median of sufficient width for pedestrians to wait and the signals are pedestrian actuated, an additional pedestrian detector shall be provided in the median. The use of additional pedestrian detectors on islands or medians where a pedestrian might become stranded should be considered.

If used, a pilot light or other means of indication installed with a pedestrian pushbutton shall not be illuminated until actuation. Once it is actuated, it shall remain illuminated until the pedestrian's green or WALKING PERSON (symbolizing WALK) signal indication is displayed.

4.12.5 Accessible Pedestrian Signal Detectors

An accessible pedestrian signal detector is a device designated to assist the pedestrian who has visual or physical disabilities in activating the pedestrian phase. Accessible pedestrian signal detectors may be pushbuttons or passive detection devices. Pushbutton locator tones may be used with accessible pedestrian signals.

At accessible pedestrian signal locations with pedestrian actuation, each pushbutton shall activate both the walk interval and the accessible pedestrian signals. At accessible pedestrian signal locations, pushbuttons should clearly indicate which crosswalk signal is actuated by each pushbutton. At corners of signalized locations with accessible pedestrian signals where two pedestrian pushbuttons are provided, a distance of at least 10 feet should separate the pushbuttons. This enables pedestrians who have visual disabilities to distinguish and locate the appropriate pushbutton.

Pushbuttons for accessible pedestrian signals should be located:

1. Adjacent to a level all-weather surface to provide access from a wheelchair and where there is an all-weather surface, wheelchair accessible route to the ramp;
2. Within 5 feet of the crosswalk extended;
3. Within 10 feet of the edge of the curb, shoulder or pavement and
4. Parallel to the crosswalk to be used.

When used, pushbutton locator tones shall be easily locatable, shall have duration of 0.15 seconds or less and shall repeat at 1-second intervals. Pushbutton locator tones should be deactivated during flashing operation of the traffic control signal.

4.13 LOCATION AND CONFIGURATION OF THE CONTROLLER AND CABINET

The following design guidelines are offered with respect to the controller and cabinet:

1. When a new controller cabinet is specified, it should be located in a protected area with reasonable set back from travel lanes. Controller cabinets should be located outside the clear zone on rural and urban flush shoulder projects and on urban curb or curb and gutter projects they should be located a minimum of 4 ft. from face of outside curb and outside the sidewalk.
2. The cabinet should be oriented such that the door faces away from the intersection to provide maximum protection for service personnel.
3. Two spare conduits to nearest pull box should be provided for future use.
4. The ***FDOT Specifications*** require that 40 feet of ground rod be installed for all controllers and that the grounding assembly for the controller must be kept at a distance greater than 6 feet from any other grounding assembly.

4.14 LOCATION AND CONFIGURATION OF SIGNAL DISPLAYS

The display design must conform to the requirements of the ***MUTCD***. Local practices will frequently dictate the choice of the options in cases where the ***MUTCD*** provides flexibility. The ***MUTCD*** requirements are reflected in the ***Design Standards***.

4.14.1 Vehicular Signal Displays

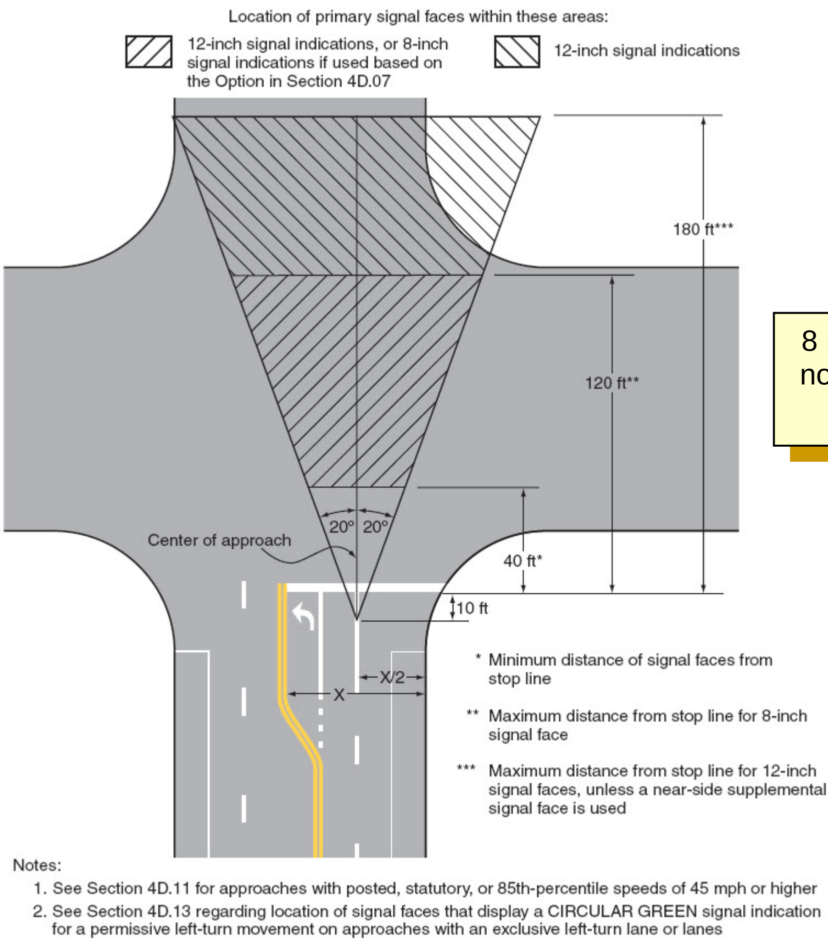
The primary design criteria with respect to vehicular signal displays include:

1. Location of Signal Faces;
2. Horizontal or vertical mounting;
3. Size of lenses (12 inch lenses are required in Florida); and,
4. Indications for each face (solid or arrow).

The ***MUTCD*** requirements are summarized as follows:

1. Location of Signal Faces: If a signalized through movement exists on an approach, a minimum of two primary signal faces shall be provided for the through movement. If a signalized through movement does not exist on an approach, a minimum of two primary signal faces shall be provided for the signalized turning movement that is considered to be the major movement from the approach.

Except where the width of an intersecting roadway or other conditions make it physically impractical, at least one and preferably both of the primary signal faces shall be located to conform to the positioning requirements shown in ***Figure 4-7*.**



8 inch signal lenses are no longer allowed in the State of Florida

Figure 4-7 Lateral and Longitudinal Location of Primary Signal Faces

In addition to the sight distance requirements for vehicles at the stop bar, the two primary signal faces required as a minimum for each approach should be continuously visible to traffic approaching the traffic control signal, from a point at least the minimum sight distance provided in ***Figure 4-8*** in advance of and measured to the stop line. This range of continuous visibility should be provided unless precluded by a physical obstruction or unless another signalized location is within this range. If the minimum sight distance in ***Figure 4-8*** cannot be met, a sign shall be installed to warn approaching

85th-Percentile Speed	Minimum Sight Distance
20 mph	175 feet
25 mph	215 feet
30 mph	270 feet
35 mph	325 feet
40 mph	390 feet
45 mph	460 feet
50 mph	540 feet
55 mph	625 feet
60 mph	715 feet

Note: Distances in this table are derived from stopping sight distance plus an assumed queue length for shorter cycle lengths (60 to 75 seconds).

Figure 4-8 Minimum Sight Distance for Signal Visibility

traffic of the traffic control signal.

The bottom of the signal housing and any related attachments to a vehicular signal face located over any portion of a highway that can be used by motor vehicles shall be at least 17.5 feet but not more than 19 feet above the pavement.

The bottom of the signal housing (including brackets) of a vehicular signal face that is vertically arranged and not located over a roadway:

a. Shall be a minimum of 8 feet and a maximum of 19 feet above the sidewalk or, if there is no sidewalk, above the pavement grade at the center of the roadway.
b. Shall be a minimum of 4.5 feet and a maximum of 19 feet above the median island grade of a center median island if located on the near side of the intersection.

The bottom of the signal housing (including brackets) of a vehicular signal face that is horizontally arranged and not located over a roadway:

a. Shall be a minimum of 8 feet and a maximum of 22 feet above the sidewalk or, if there is no sidewalk, above the pavement grade at the center of the roadway.
b. Shall be a minimum of 4.5 feet and a maximum of 22 feet above the median island grade of a center median island if located on the near side of the intersection.

2. Horizontal or vertical mounting: The decision to mount signal faces vertically or horizontally is primarily a local preference, or may be necessitated by geometric conditions. Signal faces that are located downstream of a bridge may be mounted horizontally to provide required vertical clearance while maintaining longitudinal visibility of all of the indications. Horizontally mounted heads may also provide a smaller wind profile that could reduce stress during severe weather events. Regardless of the mounting orientation, the requirements for order of the indications in the face are presented in ***Figure 4-9***.

Horizontal Mounting (Left to Right)	**Vertical Mounting** (Top to Bottom)
Circular Red	Circular Red
Left-Turn Red Arrow	Left-Turn Red Arrow
Right-Turn Red Arrow	Right-Turn Red Arrow
Circular Yellow	Circular Yellow
Left-Turn Yellow Arrow	Circular Green
Left-Turn Green Arrow	Straight-Through Green Arrow
Circular Green	Left-Turn Yellow Arrow
Straight-Through Green Arrow	Left-Turn Green Arrow
Right-Turn Yellow Arrow	Right-Turn Yellow Arrow
Right-Turn Green Arrow	Right-Turn Green Arrow

Figure 4-9 Order of Signal Lenses in Horizontally and Vertically Mounted Heads.

3. Size of lenses: Twelve-inch signal indications shall be used for all signal sections in all new signal faces. Eight-inch circular signal indications may be used in new signal faces only for:
   a. The green or flashing yellow signal indications in an emergency-vehicle traffic control signal;
   b. The circular indications in signal faces controlling the approach to the downstream location where two adjacent signalized locations are close to each other and it is not practical because of factors such as high approach speeds, horizontal or vertical curves, or other geometric factors to install visibility-limited signal faces for the downstream approach;
   c. The circular indications in a signal face that is located less than 120 feet from the stop line on a roadway with a posted or statutory speed limit of 30 mph or less;
   d. The circular indications in a supplemental near-side signal face:
   e. The circular indications in a supplemental signal face installed for the sole purpose of controlling pedestrian movements rather than vehicular movements; and
   f. The circular indications in a signal face installed for the sole purpose of controlling a bikeway or a bicycle movement.

   Existing 8-inch circular signal indications that are not included in Items a through f above, may be retained for the remainder of their useful service life.

4. Indications for each face: Each vehicular signal face shall have three, four or five signal sections. Single-section signal faces are only permitted for continuously

illuminated GREEN ARROW indications. The proper relative positions (order) of signal lenses within the signal face are shown in ***Figure 4-10***.

Required signal faces for through traffic on any one approach shall be located not less than 8 feet apart measured horizontally perpendicular to the approach between the centers of the signal faces.

“Protected only” left turn displays may be either red arrow lenses or solid red lenses with “Left Turn Signal” signs. There is no consistent statewide practice with respect to this choice. The increased target value of the solid red lens is generally recognized as an advantage that must be weighed against the increased sign clutter and wind loading. Red arrows should be used at all locations that require another sign (e.g., U Turn prohibition) in substantially the same position.

4.14.2 Supplemental Signal Faces

Supplemental signal faces should be used if engineering judgment has shown that they are needed to achieve intersection visibility both in advance and immediately before the signalized location. If used, they should be located to provide optimum visibility for the movement to be controlled. If the sight distance to the signal faces for an approach is limited by horizontal or vertical alignment, supplemental signal faces aimed at a point on the approach at which the signal indications first become visible may be used. If supplemental signal faces are used, left-turn arrows shall not be used in near-right signal faces and right-turn arrows shall not be used in far-left signal faces.

> The primary consideration in signal face placement, aiming and adjustment is to optimize the visibility of signal indications to approaching traffic
>
> ***{MUTCD}***

Near-side signal faces should be located as close as practical to the stop line. If a signal face controls a specific lane or lanes of approach, its position should make it readily visible to road users making that movement.

4.14.3 Visibility, Shielding and Positioning of Signal Faces

Signal visors should be used on signal faces to aid in directing the signal indication specifically to approaching traffic, as well as to reduce "sun phantom," which can result when external light enters the lens. The use of signal louvers for this purpose is discouraged because of the reduction in light output caused by signal louvers.

A signal backplate for target value enhancement should be used on signal faces viewed against a bright sky or bright or confusing backgrounds. The use of backplates enhances the contrast between the traffic signals and their surroundings for both day and night conditions, which is also helpful to elderly drivers.

In cases where irregular street design imposes a comparatively small angle between the orientations of signal lenses, each signal lens should, to the extent practical, be shielded or directed by signal visors, signal louvers or other means. Signal visors exceeding 12 inches in length shall not be used on free-swinging signal heads. Pedestrian movements crossing all approaches with shielded vehicular signals should be controlled by separate pedestrian signals.

> Visibility-limited signal faces shall be adjusted so bicyclists for whom the indications are intended can see them. If the signal faces cannot be aimed to serve the bicyclist, then separate signal faces shall be provided for the bicyclist.
> ***{MUTCD}***

If a signal is operated in the flashing mode for nighttime operation and the signal indication is so bright as to cause excessive glare, some form of automatic dimming should be used to reduce the brilliance of the signal indication.

The inside of signal visors (hoods), the entire surface of louvers and fins and the front surface of backplates shall have a matte black finish to minimize light reflection and to increase contrast between the signal indication and its background. To obtain the best possible contrast with the visual background, signal housings should be highway yellow.

4.14.4 Pedestrian Signal Heads

Pedestrian signal heads provide special types of traffic signal indications exclusively intended for controlling pedestrian traffic. These signal indications consist of the illuminated symbols of a WALKING PERSON (symbolizing WALK) and an UPRAISED HAND (symbolizing DONT WALK). The minimum size of pedestrian signals shall be 12 inches. All pedestrian signal heads shall include a pedestrian change interval countdown display.

Pedestrian signal heads **shall** be used in conjunction with vehicular traffic control:

1. If a traffic control signal is justified by an engineering study and meets either Warrant 4, Pedestrian Volume or Warrant 5, School Crossing;
2. If an exclusive signal phase is provided or made available for pedestrian movements in one or more directions, with all conflicting vehicular movements being stopped;
3. At an established school crossing at any signalized location; or
4. Where engineering judgment determines that multi-phase signal indications (as with split-phase timing) would tend to confuse or cause conflicts with pedestrians using a crosswalk guided only by vehicular signal indications.

Pedestrian signal heads **should** be used:

1. If it is necessary to assist pedestrians in deciding when to begin crossing the roadway in the chosen direction or if engineering judgment determines that pedestrian signal heads are justified to minimize vehicle-pedestrian conflicts;
2. If pedestrians are permitted to cross a portion of a street, such as to or from a median of sufficient width for pedestrians to wait, during a particular interval but are not permitted to cross the remainder of the street during any part of the same interval; or
3. If no vehicular signal indications are visible to pedestrians or if the vehicular signal indications that are visible to pedestrians starting or continuing a crossing provide insufficient guidance for them to decide when it is reasonably safe to cross, such as on one-way streets, at "T" intersections or at multiphase signal operations.

Pedestrian signal indications should be conspicuous and recognizable to pedestrians at all distances from the beginning of the controlled crosswalk to a point 10 feet from the end of the controlled crosswalk during both day and night. For crosswalks where the pedestrian enters the crosswalk more than 100 feet from the pedestrian signal head indications, the symbols should be at least 9 inches high.

Pedestrian signal heads shall be mounted with the bottom of the signal housing including brackets not less than 7 feet nor more than 10 feet above sidewalk level and shall be positioned and adjusted to provide maximum visibility at the beginning of the controlled crosswalk. If pedestrian signal heads are mounted on the same support as vehicular signal heads, there shall be a physical separation between them. Pedestrian signals should be designed for installation on the appropriate side of the support pole that provides for maximum protection against errant turning truck traffic.

4.14.5 Accessible Pedestrian Signals

> Safety considerations should include the installation, where appropriate, of accessible pedestrian signals that provide information in non-visual format (such as audible tones, verbal messages, and/or vibrating surfaces).
>
> ***{MUTCD}***

The primary technique that pedestrians who have visual disabilities use to cross streets at signalized intersections is to initiate their crossing when they hear the traffic in front of them stop and the traffic along side them begin to move, corresponding to the onset of the green interval. This technique is effective at the vast majority of signalized intersections. The existing environment is often sufficient to provide the information that pedestrians who have visual disabilities need to operate safely at a signalized intersection. Therefore, the vast majority of signalized intersections will not require any accessible pedestrian signals.

If a particular signalized intersection presents difficulties for pedestrians who have visual disabilities to cross safely and effectively, the underlying safety and effectiveness concerns for all pedestrians should first be addressed before considering any access issues for pedestrians who have visual disabilities. Once a particular signalized intersection is reviewed for pedestrian safety in general, an examination should consider whether accessible pedestrian signals are necessary to provide information that is not readily apparent in the existing environment.

> When used, accessible pedestrian signals shall:
> 1. Be used in combination with pedestrian signal timing.
> 2. Clearly indicate which pedestrian crossing is served by each device.
> 3. Not be limited in operation by the time of day or day of week.
>
> ***{MUTCD}***

The factors that might make crossing at an intersection difficult for pedestrians who have visual disabilities include: increasingly quiet cars, right turn on red (which masks the beginning of the through phase), continuous right-turn movements, complex signal operations, complex intersection geometry, traffic circles and wide streets. Further, low traffic volumes might make it difficult for pedestrians who have visual disabilities to discern signal phase changes.

Local organizations, providing support services to pedestrians who have visual and/or hearing disabilities, can often act as important advisors to the traffic engineer when consideration is being given to the installation of devices to assist such pedestrians. Additionally, orientation and mobility specialists or similar professionals also might be able to provide a wide range of advice..

Advice from organizations that represent pedestrians who have disabilities should be given deference because such organizations are the representative voice of the affected individuals. Agreement among such organizations should be widespread to determine that there is a community demand for the installation of accessible pedestrian signals.

The installation of accessible pedestrian signals at signalized intersections should be based on an engineering study, which should consider the following factors:

1. Potential demand for accessible pedestrian signals;
2. A request for accessible pedestrian signals;
3. Traffic volumes during times when pedestrians might be present; including periods of low traffic volumes or high turn-on-red volumes;
4. The complexity of traffic signal phasing;
5. The complexity of intersection geometry and
6. Technology that provides different sounds for each non-concurrent signal phase has frequently been found to provide ambiguous information.

The ***MUTCD*** describes and sets forth the following requirements for accessible pedestrian signals and detectors:

Accessible pedestrian signals shall have both audible and vibrotactile walk indications. Vibrotactile walk indications shall be provided by a tactile arrow on the pushbutton that vibrates during the walk interval. Accessible pedestrian signals shall have an audible walk indication during the walk interval only. The audible walk indication shall be audible from the beginning of the associated crosswalk. The accessible walk indication shall have the same duration as the pedestrian walk signal except when the pedestrian signal rests in walk.

Audible Tone Signals

Where two accessible pedestrian signals are separated by a distance of at least 10 feet, the audible walk indication shall be a percussive tone. Where two accessible pedestrian signals on one corner are not separated by a distance of at least 10 feet, the audible walk indication shall be a speech walk message. Audible tone walk indications shall repeat at eight to ten ticks per second. Audible tones used as walk indications shall consist of multiple frequencies with a dominant component at 880 Hz.

When accessible pedestrian signals have an audible tone(s), the volume of audible walk indications and pushbutton locator tones should be set to be a maximum of 5 dBA louder than ambient sound, except when audible beaconing is provided in response to an extended pushbutton press. Automatic volume adjustment in response to ambient traffic sound level shall be provided up to a maximum volume of 100 dBA.

The sound level of audible walk indications and pushbutton locator tones should be adjusted to be low enough to avoid misleading pedestrians who have visual disabilities when the following conditions exist:

1. Where there is an island that allows unsignalized right turns across a crosswalk between the island and the sidewalk.
2. Where multi-leg approaches or complex signal phasing require more than two pedestrian phases, such that it might be unclear which crosswalk is served by each audible tone.
3. At intersections where a diagonal pedestrian crossing is allowed, or where one street receives a WALKING PERSON (symbolizing WALK) signal indication simultaneously with another street.

To enable pedestrians who have visual disabilities to distinguish and locate the appropriate pushbutton at an accessible pedestrian signal location, pushbuttons shall clearly indicate by means of tactile arrows which crosswalk signal is actuated by each pushbutton. Tactile arrows shall be located on the pushbutton, have high visual contrast (light on dark or dark on light), and shall be aligned parallel to the direction of travel on the associated crosswalk. An accessible pedestrian pushbutton shall incorporate a locator tone. A pushbutton locator tone is a repeating sound that informs approaching pedestrians that a pushbutton to actuate pedestrian timing or receive additional information exists, and that enables pedestrians with visual disabilities to locate the pushbutton.

Speech Pushbutton Information Messages

Speech pushbutton information messages may provide intersection identification, as well as information about unusual intersection signalization and geometry, such as notification regarding exclusive pedestrian phasing, leading pedestrian intervals, split phasing, diagonal crosswalks, and medians or islands.

If speech pushbutton information messages are made available by actuating the accessible pedestrian signal detector, they shall only be actuated when the walk interval is not timing. They shall begin with the term “Wait,” followed by intersection identification information modeled after: “Wait to cross Broadway at Grand.” If information on intersection signalization or geometry is also given, it shall follow the intersection identification information.

Vibrotactile Signals

Vibrotactile walk indications shall be provided by a tactile arrow on the pushbutton that vibrates during the walk interval. To enable pedestrians who have visual disabilities to distinguish and locate the appropriate pushbutton at an accessible pedestrian signal location, pushbuttons shall clearly indicate by means of tactile arrows which crosswalk signal is actuated by each pushbutton. Tactile arrows shall be located on the pushbutton, have high visual contrast (light on dark or dark on light), and shall be aligned parallel to the direction of travel on the associated crosswalk.

4.14.6 Intersection Control Beacons

Intersection Control Beacons consist of one or more signal faces directed toward each approach to an intersection. Each signal face consists of one or more signal sections of a standard traffic signal face, with flashing circular yellow or circular red signal indications in each signal face. They shall be used only at an intersection to control two or more directions of travel.

Application of Intersection Control Beacon signal indications shall be limited to:

1. Yellow on one route (normally the major street) and red for the remaining approaches;
2. Red for all approaches (if the warrant for a multi-way stop is satisfied) and
3. Flashing yellow signal indications shall not face conflicting vehicular approaches.

A STOP sign shall be used on approaches to which a flashing red signal indication is shown on an Intersection Control Beacon.

If two horizontally aligned red signal indications are used on an approach for an Intersection Control Beacon, they shall be flashed simultaneously to avoid being confused with grade crossing flashing-light signals. If two vertically aligned red signal indications are used on an approach for an Intersection Control Beacon, they shall be flashed alternately.

An Intersection Control Beacon should not be mounted on a pedestal in the roadway unless the pedestal is within the confines of a traffic or pedestrian island.

Supplemental signal indications may be used on one or more approaches in order to provide adequate visibility to approaching road users. Intersection Control Beacons may be used at intersections where traffic or physical conditions do not justify conventional traffic control signals but crash rates indicate the possibility of a special need. An Intersection Control Beacon is generally located over the center of an intersection; however, it may be used at other suitable locations.

Intersection control beacons are exempt from the FDOT requirement that all signals within ten miles of the coastline be installed on mast arms.

4.14.7 Other Traffic Signal Applications

The ***MUTCD*** prescribes additional applications for traffic signals including pedestrian hybrid beacons; emergency vehicle signals; one-lane, two-way operations; freeway entrance ramps (metering); movable bridges; toll plazas; lane use; and in-roadway lights.

4.14.8 Stop Signs at Signalized Intersections

The ***MUTCD*** prescribes that STOP signs shall not be used in conjunction with any traffic control signal operation, except when:

1. The signal indication for an approach is a flashing red at all times.
2. A minor street or driveway is located within or adjacent to the area controlled by the traffic control signal, but does not require separate traffic signal control because an extremely low potential for conflict exists.
3. If a channelized turn lane is separated from the adjacent travel lanes by an island and the channelized turn lane is not controlled by a traffic control signal.

4.15 EQUIPMENT REQUIRED TO SUPPORT THE SIGNAL DISPLAYS

Signal displays may be mounted either on mast arms or span wires. It is a requirement in Florida that all signal displays within 10 miles of the coastline be mounted on mast arms because of the susceptibility of span wire mounting to high winds. Flashing beacon displays are exempt from this requirement. Section 3.5 of the FDOT Traffic Engineering Manual contains maps of the 10 mile boundary. Typical horizontal (mast arm) and vertical (span-wire) installations are shown in ***Figure 4-10***.

Figure 4-10 Typical Horizontal and Vertical Signal Installations

The following provisions in the ***Design Standards*** apply to the supporting equipment and structures for signal displays:

1. ***Index 17723 and 17725*** set forth the FDOT design and installation requirements for steel and concrete poles, respectively.
2. ***Index 17727*** *Signal cable and span wire installation details* specify installation, suspension and grounding methods for signals on span-wires. A typical example of a span wire mounted signal is shown in ***Figure 4-10***.
3. ***Index 17733*** *Aerial interconnect* specifies cable drop and termination details for overhead interconnect.
4. ***Index 17736*** *Electric power service* specifies equipment, dimensions, conduit and grounding requirements for connection of the controller box to the power source.
5. ***Index 17743 and 17745*** provide instructions, examples, component data and installation requirements for mast arms.

All new signals installed by the Department on the State Highway System that are within ten miles of the coastline shall be supported by mast arms with signal heads rigidly attached to the mast arm.

{PPM}

6. ***Index 17748*** provides details for Free-Swinging Internally-Illuminated Street Sign Assemblies required at every traffic signal installation.
7. ***Index 17764*** *Pedestrian control signal installation details* specifies pole types, anchors, conduit installation and grounding methods for pole-mounted installation.
8. ***Index 17781*** *Vehicle loop installation details* deals primarily with lead-in connections from loops installed in the roadway. Specifies splicing procedures, gutter crossings, etc.
9. ***Index 17784*** *Pedestrian detector assembly installation details* specify the location of pedestrian detectors and the requirements for advisory signs giving directions to pedestrians.
10. ***Index 17841*** *Cabinet installation details* specify cabinet types, installation and grounding details for pole-mounted and pedestal-mounted controller cabinets.

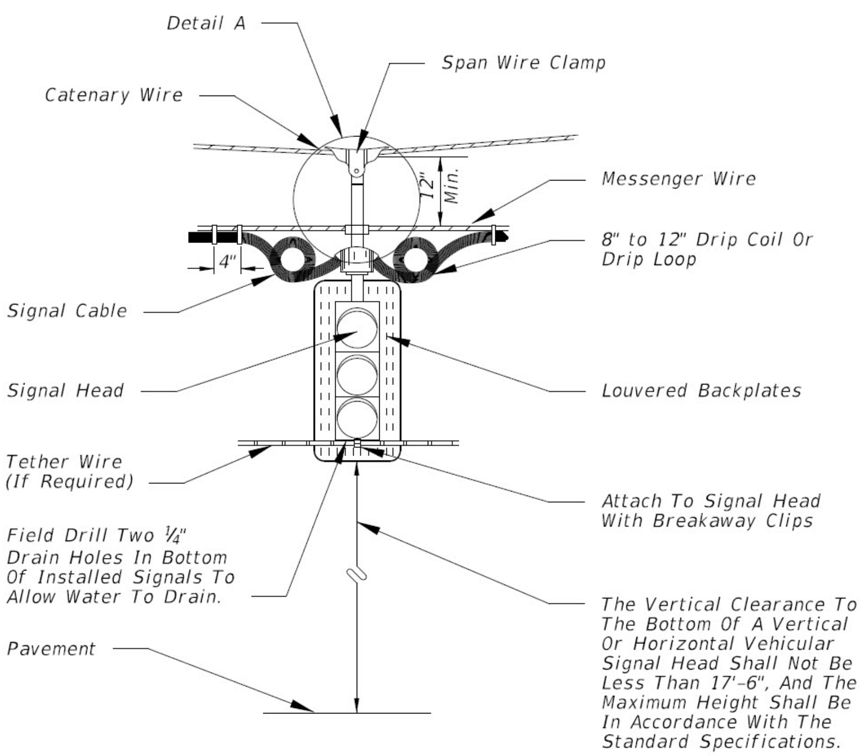


Figure 4-11 Typical Span Mounted Signal Head as Shown in *Design Standards*, *Index 17727*

The following additional guidance is offered with respect to signal mounting equipment:

> The design of traffic signal mast arms and foundations shall be included in the plans.
> ***{PPM}***

1. In general, mast arms with horizontal signal heads should be parallel to their intended stop bar.
2. The elevation of the base for poles supporting mast arms is a critical element in the construction because it determines the final mounting height of all signal displays.
3. Horizontal signal heads must be mounted at approximately the same elevation as the horizontal mast arm member. Signal heads mounted below the mast arm to remedy elevation problems are not acceptable.
4. Mast arms should be of sufficient length and design to support placement of additional heads and signs that may be deemed necessary in the future.

The following items should be considered when placing signal supports and cabinets:

> Signal poles shall not be located in medians.
> ***{PPM}***

1. Reference should be made to ***PPM*** Volume I, Chapter 8, Section 8.3.2 and to the Americans with Disabilities Act.
2. Signal supports shall not be located in medians. On rural and urban facilities with flush shoulders they should be located outside the clear zone. On urban curb or curb and gutter facilities placement should be 4 feet from the face of outside curbs and outside the sidewalk. However, when necessary, the Signal Poles may be located within sidewalks as long as an unobstructed sidewalk width of 4 feet or more (not including the width of the curb) is provided.
3. No part of a concrete base for a signal support should extend above the ground level at any point.
4. A signal support or controller cabinet should not obstruct the sidewalk or access from the sidewalk to the crosswalk.

This chapter has set forth the principles and criteria for signal design that are recognized by the FDOT and identified the design methodology that is commonly used throughout the state. The next chapter will do the same thing for traffic signs and markings.

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Chapter 5

Signs and Markings

Tables

Figures

5 SIGNS AND MARKINGS

5.1 GENERAL APPLICATION

Nearly all highway projects require the use of traffic signs and pavement markings. The main purpose of such signs and markings is to present information to road users. There are many criteria and standards to be observed because of the need for clarity and uniformity in the way information is presented. This chapter addresses signing and marking requirements for at-grade intersections.

5.2 SIGNIFICANT REFERENCES

The following referenced documents govern the design of signs and markings for intersections in Florida.

1. The ***Manual on Uniform Traffic Control Devices (MUTCD)***, as described in Chapter 1, establishes national standards that promote uniformity and facilitate driver comprehension of traffic control devices***.*** Part 2 of the ***MUTCD*** describes signs and Part 3 covers markings.

2. The ***Design Standards*** set forth requirements that apply specifically to the design and placement of signs and markings at intersections. This document includes the following relevant sections:

   Index 11200 Multi-Column Ground Signs,
   Index 11860 Single Column Ground Signs,
   Index 17302 Typical Sections for Placements of Single and Multi-Column Signs,
   Index 17344 School Signs and Markings,
   Index 17346 Special Marking Areas,
   Index 17349 Traffic Controls for Street Terminations,
   Index 17352 Typical Placement of Reflective Pavement Markers,
   Index 17355 Special Sign Details and
   Index 17748 Free-Swinging, Internally-Illuminated Street Sign Assemblies.

   Note: The Index Sheets that specifically deal with sign structures and mountings are not included in this list.

3. The FHWA Manual ***Standard Highway Signs*** provides information on sizes and color specifications for road signs.

4. The FHWA ***Standard Alphabets for Signs and Markings*** provides font types and dimensions of letters to be used on road signs.

5. The Florida ***Traffic Engineering Manual (TEM)*** contains sections that describe the proper use of selected signs and markings appropriate for use at intersections. Particular attention should be given to recommended sign sizes for

the accommodation of elder drivers. The sizes generally exceed the minimum sizes specified in the ***MUTCD***.

6. The ***FDOT Specifications*** contain sections that describe selection and use of appropriate materials for signs and markings.
7. The FHWA ***Older Driver Highway Design Handbook***, (Washington, D.C., 1998), provides specific guidance, taking into account the special needs of older drivers.
8. The ***AASHTO, Guide for Development of Bicycle Facilities***, Sixth Edition, (Washington, D.C., 2011), provides updated criteria for the design, construction and operation of bikeways.

5.3 SIGNING AND MARKING MATERIALS

The choice of materials is an essential part of the design of intersection signs and markings. Only materials that have been certified or approved by the FDOT may be used for such signs and markings.

5.3.1 Sign Materials

Standard sign panel messages shall be fabricated in accordance with details included in the *Standard Highway Signs Manual*. Retroreflective sheeting shall be applied to these panels with mechanical equipment and in a manner specified for the manufacture of traffic control signs by the sheeting manufacturer. Additional details of sign construction are given in the ***FDOT Specifications*****.**

5.3.2 Marking Materials

The most common materials for pavement markings are paint and thermoplastic. Thermoplastic may be called for in plans on those projects that are exclusively concrete pavement surfaces. Pavement marking material on projects that include new asphalt surfaces will generally be paint, rather than thermoplastic. This is based on the requirement of a 90-day curing period for new asphalt. Thermoplastic markings on these projects must be placed using a separate contract from the contract for constructing the pavement surfaces. Materials for paint and thermoplastic are covered by the ***FDOT Specifications,*** Section 971.

5.4 GENERAL REQUIREMENTS FOR SIGNS AND MARKINGS

Parts 2 and 3 of the ***MUTCD*** prescribe requirements governing the shapes, sizes, colors, placement and other design details for signs and markings appropriate for intersections. Florida guidelines and practice go well beyond the national ***MUTCD*** provisions. In addition there are specific requirements that apply to individual intersection elements such as approaches, medians and crosswalks. The general requirements are covered first.

Regulatory, warning and guide signs are used in various combinations at intersections. Both changeable message and static signs may be used at these locations. Care should be taken to add signs when warranted and not to install excessive numbers of signs.

5.4.1 General Requirements for Signs

Retro-reflectivity of traffic signs greatly increases their nighttime visibility. Accordingly, retro reflectivity promotes efficient traffic flow, driving comfort and highway safety. However, retro reflective materials degrade with the effects of traffic and weather. Therefore, these signs should be evaluated on a periodical schedule.

5.4.2 General Requirements for Markings

The primary function of highway and street markings is to define the operating areas for vehicles. Markings are used to convey regulatory information, driver guidance and warnings to road users. The major marking types include pavement and curb markings, object markers, delineators, colored pavements, barricades, channelizing devices and islands. Markings are used to supplement other traffic control devices such as signs, signals and other markings. The visibility of the markings can be limited by water, debris and snow on or near the markings. The durability of markings is affected by traffic, weather and the inherent characteristics of the marking materials.

The effectiveness of pavement markings can be enhanced by the addition of audible and tactile features such as rumble strips, raised pavement markers or other devices that may alert the road user that a boundary of the roadway is being traversed.

Typical markings prescribed by the ***MUTCD*** for intersections are shown in ***Figure 5-1***.

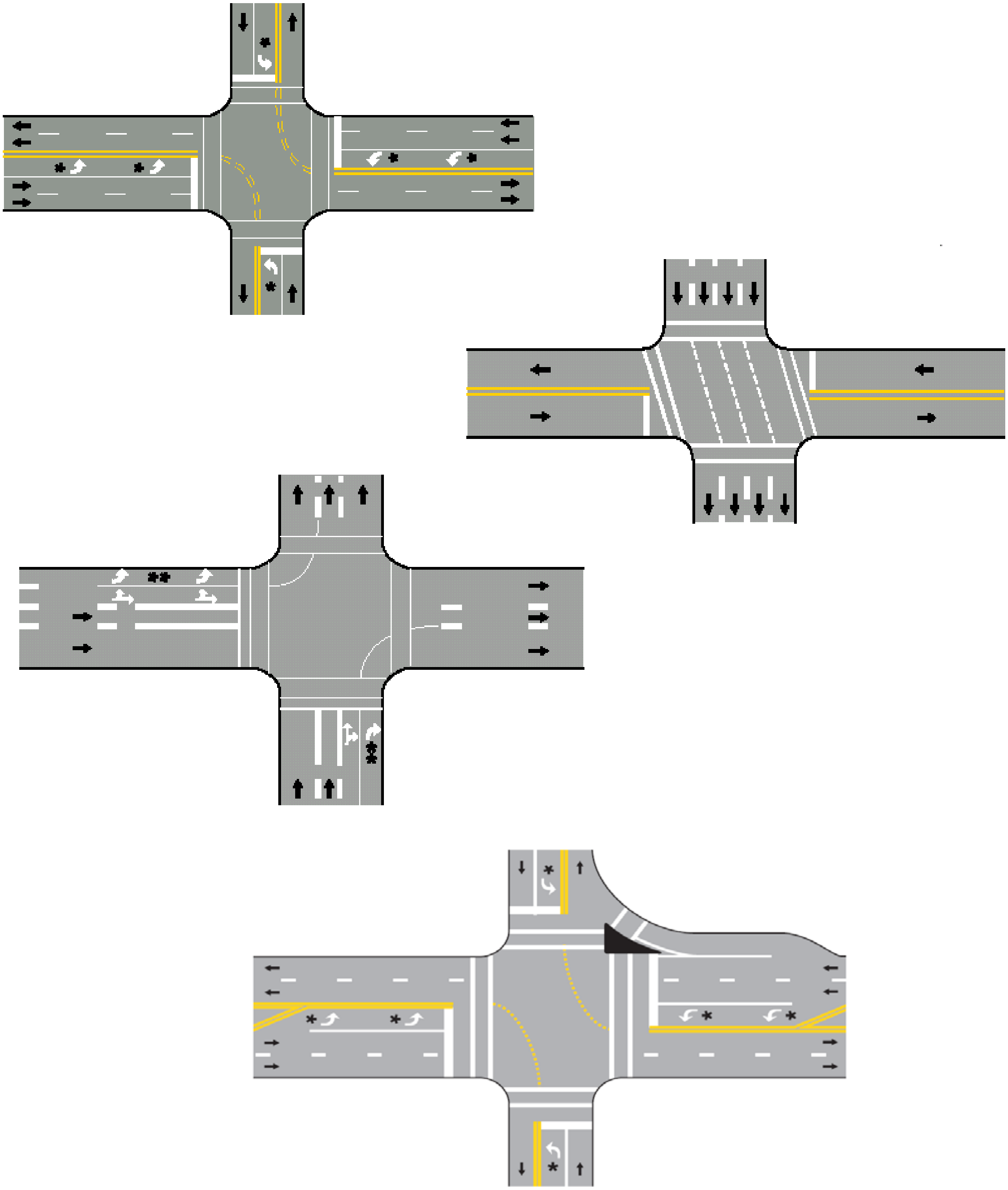


Figure 5-1 Typical Markings for Four Intersection Types (*MUTCD)*

5.4.3 Special Marking Requirements for Bicycle Lanes

Bicycle lane striping should not be continued through street intersections or across pedestrian crosswalks, even if there are no painted crosswalks. The bicycle lane striping should stop at the near side of the cross street property line and resume at the far side property line. An exception to this requirement is the extension of dotted guidelines through particularly complex intersections. The same bicycle lane striping criteria apply regardless of whether parking is permitted in the vicinity of the intersection.

At signalized or stop controlled intersections with right-turning motor vehicles, the solid striping to the approach should be replaced with a dotted line with 2 foot lines and 4 feet spaces. The length of the dotted line section is normally 50 feet to 200 feet.

For unsignalized intersections with low volumes of right-turning motor vehicles and no stop controls, solid bicycle lane striping should continue to the crosswalk on the near side of the intersection. At a location with a bus stop, the 6-inch solid line should be replaced with a dotted line with 2 feet lines and 4 feet spaces for the length of the bus stop. The bicycle lane striping should resume at the outside line of the crosswalk on the far side of the intersection. Details for bicycle lane striping are presented in ***Design Standards***, ***Index 17347***.

If a bus stop is located on a far side of the intersection rather than on a near side approach, the solid white line should be replaced with a dotted line for a distance of at least 260 feet from the crosswalk on the far side of the intersection.

At “T” intersections with no painted crosswalks, the bike lane striping on the side across from the “T” intersection should continue through the intersection area with no break. If there are painted crosswalks, the bike lane striping on the side across from the “T” should be discontinued only at the crosswalks.

5.5 APPROACH SIGNS AND MARKINGS

The driver must be provided with information related to navigation, orientation, guidance and restrictions at each intersection and in advance of the intersection. Navigation is provided as street signs or Router Markers. Orientation is provided through Cardinal Direction information (typically with Route Markers or Street Signs). Lane use and other regulatory signs and pavement markings provide guidance cues, as well as, restrictions or prohibitions.

5.5.1 Street Name Signs

Street name signs should be erected in urban areas at all street intersections regardless of other route marking that may be present and should be erected in rural districts to identify important roads not otherwise marked. Street name and advance street name guide signs should only be used to identify cross streets. They are not intended to identify destinations such as cities or facilities.

According to the ***MUTCD***, lettering on street name signs should be at least 6 inches high for upper-case letters and 4.5 inches for lower-case letters. The ***TEM*** extends the ***MUTCD*** criteria. Specifically, the ***TEM*** requires that lettering on street name signs be 8-inch upper case and 6-inch lower case, Series E or E Modified. Supplementary lettering to indicate the type of street (e.g., Street, Avenue, Road, etc.) or section of city (e.g., N.W.) may be in smaller lettering, at least 6 inches high. Conventional abbreviations are acceptable except for the street name itself. On multi-lane facilities with speed limits greater than 40 mph, post-mounted Street Name signs should be composed of initial upper-case letters at least 8 inches in height and lower-case letters at least 6 inches in height. If overhead Street Name signs are used, the lettering should be composed of initial upper-case letters at least 12 inches in height and lower-case letters at least 9 inches in height. Other requirements of the ***TEM*** include:

1. The word Street, Boulevard, Avenue, etc., may be abbreviated or deleted to conserve sign panel length unless confusion would result due to similar street names in the area.
2. When a cross street is known by both route number and a local name, use of the local name is preferred.
3. When a cross street has dual local street name designations, both names may be used.
4. When a cross street has a different name on each side of the intersection two signs should be used with one on the left and one on the right side of the intersection.
5. The preferred location is to the right side of the signal heads.
6. Internally-illuminated signs shall be used whenever possible to provide better night-time visibility, and to benefit older drivers. When used, the devices shall be on the Approved Products List (APL). They shall be designed using a white message on a green background, and if a border is used it shall be white.

A symbol or letter designation may be included to identify the governmental jurisdiction. If used, the length of the designation shall not exceed the height of the sign and should be positioned to the left of the street name.

5.5.2 Intersection Markings

> Advance warning signs shall be installed on an approach to a primary traffic control device that is not visible for a sufficient distance to permit the road user to respond to the device.
>
> ***{MUTCD}***

Typical markings for intersections are illustrated in ***Design Standards Index 17344 and 17346***. There are many possible geometric configurations, including intersections where vehicles in two and three lanes are permitted to make left turns. The index drawings provide the basic principles that may be applied to a wide variety of situations.

5.5.3 Advance Warning

When sight distance is limited or transitory events are likely to be encountered, warning signs should be erected prior to affected intersections. These signs should be placed along the street at a distance sufficient for the driver to perceive, identify, decide and perform necessary maneuvers. The time needed to react to these signs will generally vary between 3 and 10 seconds, depending upon the speed of the vehicle and the complexity of the intersection. All warning signs shall be a minimum of 30 x 30 inches unless otherwise specified. ***Table 5-1*** summarizes the more critical advance signs needed at intersections.

Because warning signs are erected primarily for drivers who are unfamiliar with the area and the particular road, great care must be given to the placement of such signs. Warning signs should provide adequate time for the driver to perceive, identify, decide and perform any necessary maneuver. This total time to perceive and appropriately respond to a sign is the sum of the times necessary to perceive, identify, understand, decide and execute an appropriate response. This is often referred to as Perception-Reaction Time (PRT). The PRT can vary from about 3 seconds for general warning signs to 10 seconds for high complexity conditions. ***Table 5-2*** lists suggested minimum sign placement distances that may be used for three conditions.

Other warning signs that advise of potential hazards not related to a specific location may be installed at the most appropriate locations. Minimum spacing between warning signs with different messages normally should he based on the PRT for driver comprehension and reaction. The effectiveness of the placement of any warning sign should be tested periodically under both day and night conditions.

Table 5-1 Advance Warning Signs Used at Intersections (*MUTCD*)

Sign Type	***Reference***	***Utilization***
Stop Ahead (W3-1)		Approach to Stop sign not visible at sufficient distance that allows vehicle to stop at Stop Sign; obstruction may be permanent or intermittent.
Yield Ahead (W3-2)		Approach to Yield sign not visible at sufficient distance that allows vehicle to stop at Yield Sign.
Signal Ahead (W3-3)		Approach to signal not visible at sufficient distance that allows vehicle to stop at signal.
Advance Pedestrian Crossing (W11-2)		Mid-block crossings and uncontrolled approaches to an intersection with a marked pedestrian crossing. Not used on signalized or stop controlled approaches. May also be used (per MUTCD) in combination with a diagonal arrow plaque to identify the stop line location.
Cross Road (W2-1)		Approach to obscured crossroad; used in conjunction with Junction signing or advance route turn assembly.
Roundabout (W2-6)		Approach to roundabouts.
Side Road, T Symbol or Y Symbol (W2-2, W2-3, W2-4 and W2-5	    Warns of traffic from directions indicated in symbol.	

Table 5-2 Guide for Estimating Advance Warning Sign Placement Distance

Posted or 85th-Percentile Speed	Advance Placement Distance[1]								
	Condition A: Speed reduction and lane changing in heavy traffic[2]	Condition B: Deceleration to the listed advisory speed (mph) for the condition							
		0[3]	10[4]	20[4]	30[4]	40[4]	50[4]	60[4]	70[4]
20 mph	225 ft	100 ft[6]	N/A[5]	—	—	—	—	—	—
25 mph	325 ft	100 ft[6]	N/A[5]	N/A[5]	—	—	—	—	—
30 mph	460 ft	100 ft[6]	N/A[5]	N/A[5]	—	—	—	—	—
35 mph	565 ft	100 ft[6]	N/A[5]	N/A[5]	N/A[5]	—	—	—	—
40 mph	670 ft	125 ft	100 ft[6]	100 ft[6]	N/A[5]	—	—	—	—
45 mph	775 ft	175 ft	125 ft	100 ft[6]	100 ft[6]	N/A[5]	—	—	—
50 mph	885 ft	250 ft	200 ft	175 ft	125 ft	100 ft[6]	—	—	—
55 mph	990 ft	325 ft	275 ft	225 ft	200 ft	125 ft	N/A[5]	—	—
60 mph	1,100 ft	400 ft	350 ft	325 ft	275 ft	200 ft	100 ft[6]	—	—
65 mph	1,200 ft	475 ft	450 ft	400 ft	350 ft	275 ft	200 ft	100 ft[6]	—
70 mph	1,250 ft	550 ft	525 ft	500 ft	450 ft	375 ft	275 ft	150 ft	—
75 mph	1,350 ft	650 ft	625 ft	600 ft	550 ft	475 ft	375 ft	250 ft	100 ft[6]

[1] The distances are adjusted for a sign legibility distance of 180 feet for Condition A. The distances for Condition B have been adjusted for a sign legibility distance of 250 feet, which is appropriate for an alignment warning symbol sign. For Conditions A and B, warning signs with less than 6-inch legend or more than four words, a minimum of 100 feet should be added to the advance placement distance to provide adequate legibility of the warning sign.

[2] Typical conditions are locations where the road user must use extra time to adjust speed and change lanes in heavy traffic because of a complex driving situation. Typical signs are Merge and Right Lane Ends. The distances are determined by providing the driver a PRT of 14.0 to 14.5 seconds for vehicle maneuvers (2005 AASHTO Policy, Exhibit 3-3, Decision Sight Distance, Avoidance Maneuver E) minus the legibility distance of 180 feet for the appropriate sign.

[3] Typical condition is the warning of a potential stop situation. Typical signs are Stop Ahead, Yield Ahead, Signal Ahead, and Intersection Warning signs. The distances are based on the 2005 AASHTO Policy, Exhibit 3-1, Stopping Sight Distance, providing a PRT of 2.5 seconds, a deceleration rate of 11.2 feet/second^2, minus the sign legibility distance of 180 feet.

[4] Typical conditions are locations where the road user must decrease speed to maneuver through the warned condition. Typical signs are Turn, Curve, Reverse Turn, or Reverse Curve. The distance is determined by providing a 2.5 second PRT, a vehicle deceleration rate of 10 feet/second^2, minus the sign legibility distance of 250 feet.

[5] No suggested distances are provided for these speeds, as the placement location is dependent on site conditions and other signing. An alignment warning sign may be placed anywhere from the point of curvature up to 100 feet in advance of the curve. However, the alignment warning sign should be installed in advance of the curve and at least 100 feet from any other signs.

[6] The minimum advance placement distance is listed as 100 feet to provide adequate spacing between signs.

5.5.4 Use of Supplemental Warning Plaques

Supplemental plaques should be used sparingly to reinforce advance warnings and to clarify the nature of the situation that the driver is about to encounter. The supplemental plaques that have the strongest potential application in advance of intersections include distance, speed and street name. The ***MUTCD*** prescribes minimum sizes for supplemental warning plaques. The ***MUTCD*** introduces a new use for the diagonal arrow plaque attached to the standard advance warning signs for pedestrian crossings and school crossings. This sign assembly replaces a discontinued sign for the purpose of establishing the crossing location.

> Supplemental plaques shall be used only in combination with warning or regulatory signs. They shall not be mounted or displayed alone. If used, a supplemental plaque shall be installed on the same post(s) as the warning sign.
>
> ***{MUTCD}***

A supplemental plaque shall have the same color legend, border and background as the warning sign with which it is displayed. Supplemental plaques shall be square or rectangular.

5.6 ENTRY POINT TREATMENT

For minor intersections and other intersections without channelization, there are no special entry point treatments required. The intersection of major roads having multiple lanes and wide medians (30 feet or wider) may require a channelization treatment for left-turning traffic to allow the free flow of traffic. When islands and raised medians are used, the curb nose of the island and the median within 20 feet of the point of vehicle entry should be painted with reflective paint. When channelization is implemented with paint alone, transverse lines or chevron markings should be placed in the neutral area to emphasize the path to be taken by left-turning vehicles. The color of these markings should conform to the ***MUTCD*** requirements. If traffic is moving in the same direction on both sides of the markings, the color should be white. If traffic is moving in opposite directions, then the color should be yellow. The pavement word message "ONLY" with a left-turn arrow may also be used near the entry point within the turn lane. See the discussion on the use of “ONLY” in Section 5.8.

5.6.1 Stop and Yield Line Markings

Stop lines are used at intersections for control both vehicular and non-motorized traffic. They are solid white lines, prescribed by ***Design Standards***, ***Index 17346*** to be 24 inches wide, extending across all approach lanes. They should be used in both rural and urban areas where it is important to indicate the point, behind which vehicles are required to stop, in compliance with a STOP sign, traffic signal or other enforceable requirement.

Stop lines, where used, should be placed 4 feet in advance of and parallel to the nearest crosswalk line. In the absence of a marked crosswalk, the stop line should be placed at the desired stopping point, in no case more than 30 feet nor less than 4 feet from the nearest edge of the intersecting roadway.

If a stop line is used with a STOP sign, it should be placed in line adjacent to the STOP sign. However, if the sign cannot be located exactly where vehicles are expected to stop, the stop line should be placed at the stopping point.

Yield lines may be used to indicate the point behind which vehicles are required to yield in compliance with a YIELD sign. The ***MUTCD*** prescribes a yield-line marking consisting of a series of triangles. The individual triangles comprising the yield line should have a base of 12 to 24 inches wide and a height equal to 1.5 times the base. The space between the triangles should be 3 to 12 inches. Yield lines shall always be used at the entry to roundabouts.

5.7 TRAFFIC SIGNAL SIGNS

To supplement traffic signal control, auxiliary signs are often necessary for the instruction of pedestrians, cyclists and drivers. Signal instruction signs should be located adjacent to the signal face to which they apply.

Signal instruction signs may be needed at certain locations to clarify signal control. With the exception noted below, each of the signs should be 24 × 30 inches. Among the legends for this purpose are LEFT ON GREEN ARROW ONLY, LEFT TURN YIELD ON GREEN (symbolic green ball) or LEFT (RIGHT) TURN SIGNAL for compliance with certain turn signals, STOP HERE ON RED (24 × 30 inches) for observance of signal limit lines, DO NOT BLOCK INTERSECTION for avoidance of traffic obstructions and USE LANE(S) WITH GREEN ARROW for obedience to lane-direction control signals.

> 1. Signs should be located adjacent to the signal face to which they apply.
> 2. Stop signs shall not be used in conjunction with traffic signals (with exceptions).
>
> ***{MUTCD}***

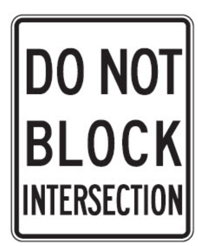
The NO TURN ON RED sign should be used to indicate that a right turn on red (or left turn on red for one-way streets) is not permitted. The NO TURN ON RED sign should have standard dimensions of 24 x 30 inches and 30 × 30 inches for word signs and symbolic signs, respectively. The sign should be erected near the appropriate signal head.

A NO TURN ON RED sign may be considered whenever an engineering study determines that one or more of the following conditions exist:

1. Sight distance to vehicles approaching from the left (or right, if applicable) is inadequate.
2. The intersection area has geometrics or operational characteristics that may result in unexpected conflicts.
3. There is an exclusive pedestrian phase.
4. Significant pedestrian conflicts are resulting from RTOR maneuvers.
5. More than three RTOR crashes per year have been identified for the particular approach.
6. There is significant crossing activity by children, elderly or disabled people.

Where improved utilization of progressive signal systems is needed, the traffic signal speed sign should be used.

5.7.1 Do Not Block Intersection



Queues at signalized intersections, may cause blockages of nearby minor streets. A regulatory DO NOT BLOCK INTERSCTION may be used to discourage this practice. The use of these signs is generally limited to local roadways. If used, these signs should be 24 × 30 inches and should be located immediately prior to the intersection.

5.7.2 Left Turn Yield on Green

Signal instruction signs are needed at certain intersections to clarify signal control. These regulatory signs may be displayed with an arrow to help drivers locate the appropriate turning lane or with a green ball when it is clear as to which lane drivers should be in prior to turning. They should be 24 × 30 inches.

5.7.3 Advance Pedestrian Crossing

Non-vehicular warning signs may be installed to warn drivers of a pedestrian crossing ahead. They are only used on approaches to mid-block crossings and marked crosswalks at intersections. They are not used on approaches with signal or stop sign control.

> Advance pedestrian signs should be used only at locations where the crossing activity is unexpected or at locations not readily apparent.

If the location of the crosswalk is not apparent and therefore requires emphasis, a second non-vehicular warning sign should be installed at the crossing with a supplemental plaque consisting of a diagonal downward arrow (W16-7P). If the crossing is not delineated by crosswalk pavement markings, then the sign and arrow are both required.

5.7.4 Guide Signs and Trail Blazers

Both guide signs and trailblazers, including emergency evacuation route signs, are used in the vicinity of intersections (450 to 900 feet in advance of the intersection) when turning is required to maintain route continuity. In general both guide signs and trailblazers should be erected at strategic locations, usually along major urban arterials, to indicate the direction to the nearest or most convenient point of access. If route continuity is maintained without turns, then neither of these sign types should be used at or near intersections.

These signs should not exceed 30 inches × 30 inches when installed at intersections. Note that larger sizes are sometimes specified for use at interchanges and other non-intersection locations. The use of the word “TO” indicates that the road or street where the marker is posted is not a part of the indicated route but that a driver is merely being directed progressively toward the route.

A trailblazer assembly shall consist of a “TO” marker, a cardinal direction marker if needed, a route marker or a special road facility symbol and a single-headed directional arrow pointed along the route leading to the facility.

5.8 TREATMENT OF TURN LANES

Where traffic volumes, roadway design or reduced visibility conditions warrant the use of right and/or left turn lanes, specific markings and signs are needed. The purpose of such markings and signs is to provide control for guiding vehicles through the intersection. A dotted line may be used to extend markings through the intersection area. Where a greater degree of restriction is required, solid lane lines or channelizing lines may be continued through intersections. A frequent use for the channelizing line is to separate turning movements.

> Where through traffic lanes approaching an intersection become mandatory turn lanes, lane-use arrow markings shall be used and shall be accompanied by standard signs.
>
> ***{MUTCD}***

It is critical that markings clearly designate the path to be taken by drivers. Arrows should be used one or more times depending upon the length of the turn lane. The ***MUTCD*** indicates that the ONLY word marking may be used to supplement lane-use arrow markings. The ***TEM*** adds the provision that the word message “ONLY” is used to supplement the arrows under the following conditions:

1. Where a movement that would otherwise be legal is to be prohibited.
2. Where unusual geometrics or alignment of an exclusive turn lane may result in driver confusion or misunderstanding.
3. Where an established through lane becomes an exclusive turn lane, the word “ONLY” shall be used with the arrow symbol indicating the required turning movement.

Figure 5-2 shows the markings for a single-lane right turn lane. ***Figure 5-3*** illustrates the proper treatment of markings for left turn lanes. Please refer to Chapter 3, Section 3.13 for a discussion of the geometric design aspects of turn lanes.

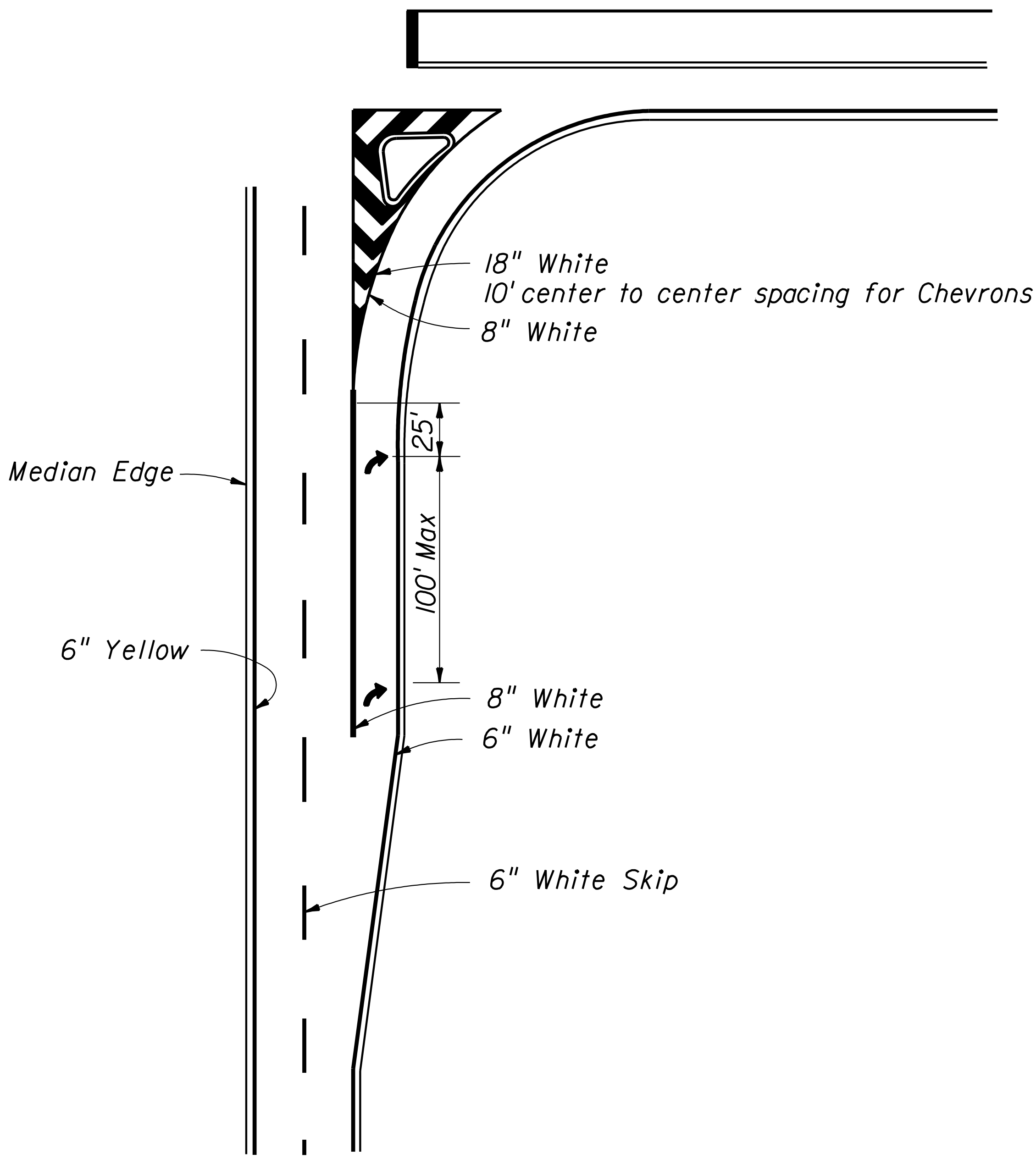


Figure 5-2 Markings and Channelization for Single-Lane Right Turn Lanes (*Design Standards*, *Index 17346*)

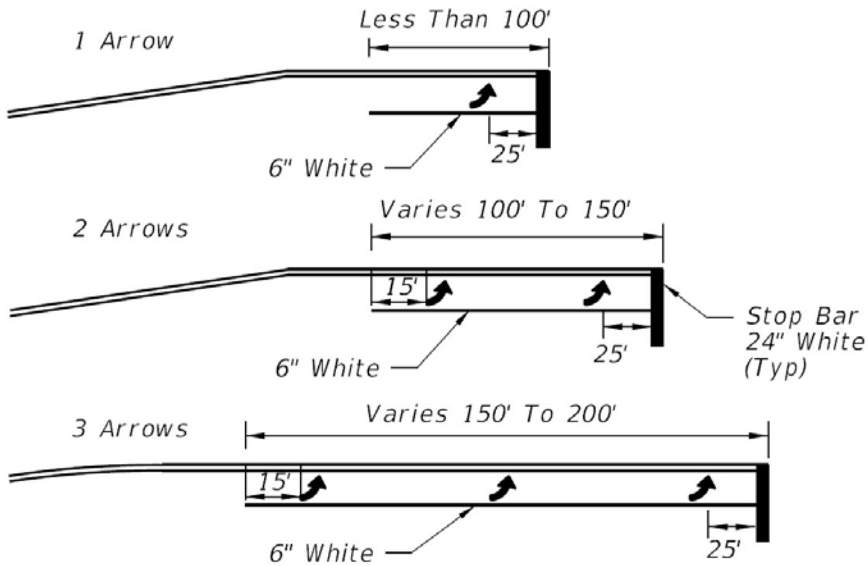


Figure 5-3 Marking of Left Turn Lanes (*Design Standards*, *Index 17346*)

Notes:

1. Yellow left-turn edge markings may be used adjacent to raised curb or grass medians if the lane use is not readily apparent to approaching drivers.
2. Arrows should be evenly spaced. Add one arrow for each 100 feet of length in excess of 200 feet.

5.9 SPECIAL PROVISIONS FOR PEDESTRIANS, BICYCLES AND SCHOOLS

Signs and markings require a somewhat different set of application rules for pedestrians and bicyclists. Drivers must however be able to react to the many markings and signs that are primarily applicable to non-motorized travelers. The sizes and content of some signs can differ because of the increasing reading time available and the reduced number of demands on pedestrians and bicyclists.

5.9.1 School Zones

Signs and markings in school zones are an example of a situation in which both motorists and non-motorized travelers must be able to read the same messages but must act differently. Drivers are met with the word message “SCHOOL” painted on the pavement surface to indicate that they must be alert and must drive more slowly during certain periods of school days. Posted speeds are reduced on arterial routes abutting school property during periods of school-related activity. The speed limit signs may be supplemented with flashing beacons for added emphasis when the reduced speed limits are in effect.

Figure 5-4 shows the recommended configuration of markings and signs for an approach to a signalized intersection. The treatment at stop-controlled crosswalks is similar, except that the crossing sign is not installed at the crosswalk. (See ***Design Standards***, ***Index 17344***.) Details of the markings at the intersection are provided in ***Design Standards***, ***Index 17346.***

Table 5-3 summarizes the various signs that are commonly used in school zones.

1. All school signs shall be retroreflectorized or illuminated.
2. School crosswalks shall be 6 feet minimum, 10 feet standard without public sidewalk curb ramps or 10 feet minimum with public sidewalk curb ramps. See ***Index 17346***.
3. For signalized intersections or mid-block signalized crossings where flashing beacon speed limit signs are installed, the minimum distance from the speed limit sign to the stop line shall be 100 feet. The sign shall not block the view of the signal.

{***Design Standards***}

The sign to be placed at the crosswalk is referred to as the "School Crosswalk Warning Assembly," and consists of a second school advance warning sign supplemented by a diagonal arrow. Note that the ***MUTCD*** specifies yellow as a standard background color for school signs with an option to use "fluorescent yellow" for certain sign types, including the two signs depicted here. However, use of the yellow-green fluorescent sheeting is required for school signs on FDOT projects in accordance with ***FDOT Specifications,*** Section 700-2.5.

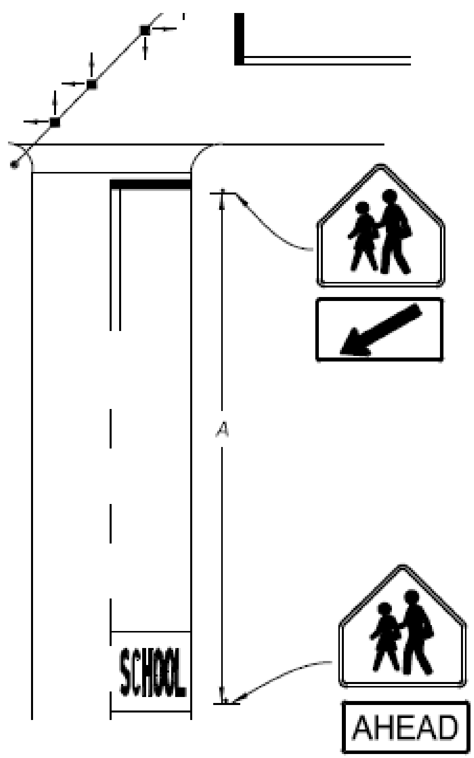


Figure 5-4 Typical Signs and Markings for a School Crosswalk at a Signalized Intersection

Table 5-3 Signs Used at Intersections in School Zones (*MUTCD*)

Sign Type	Application	Location	Sign Characteristics
School Advance Warning	Prior to locations where school grounds are adjacent to the highway or prior to school crossing	Not less than 150 ft. or more than 700 ft.	36 x 36 in. (rural areas) 30 x 30 in. (urban areas)
School Crossing Warning Assembly	At school crossings adjacent to schools and at established crossings	As close as possible to crossing	36 x 36 in. (rural areas) 30 x 30 in. (urban areas)
School Bus Stop (Warning)	Locations where students are picked up and discharged and where visibility is less than 500 ft.	Not less than 150 ft. or more than 700 ft.	36 x 36 in. (rural areas)
School Speed Limit (Regulatory)	In school areas, after engineering and traffic study	Not less than 150 ft. or more than 700 ft.	24 x 30 in. (may be used with flasher or may be changeable message)

5.9.2 Crosswalk Signs and Markings

Crosswalk lines are solid white lines marking both edges of the crosswalk. The ***MUTCD*** requires a minimum width of 6 inches. The standard width for the SHS is 12 inches. They should be spaced 6 feet apart minimum or 10 feet standard. Crosswalk lines on both sides of the crosswalk should extend across the full width of pavement to discourage diagonal walking between crosswalks.

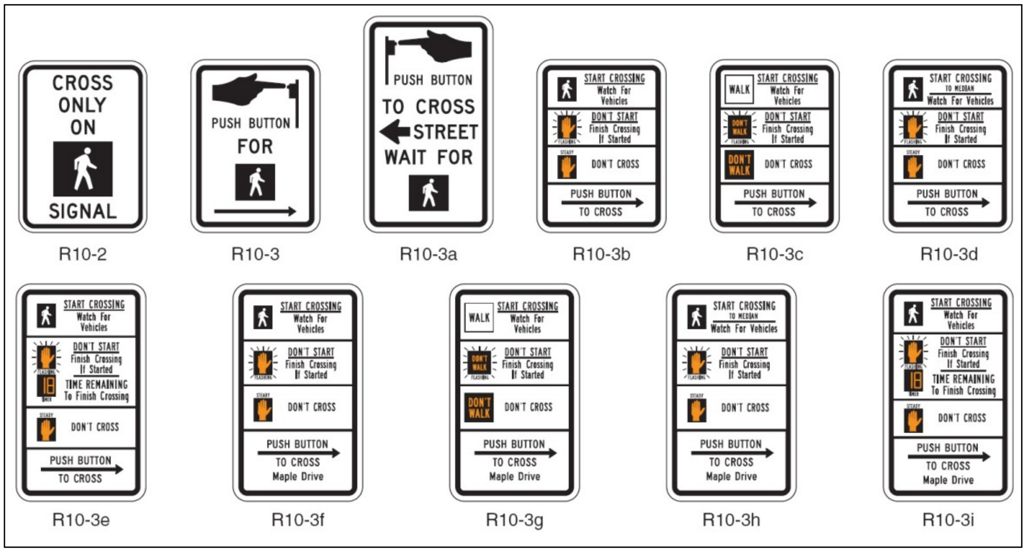
Crosswalks should be marked at all intersections on established school routes where there is substantial crossing conflicts between vehicles and students, where students are permitted to cross between intersections or where students could not otherwise be expected to recognize the proper place to cross.

Typical crosswalk markings for public sidewalk curb ramps are provided in ***Design Standards***, ***Index 17346***.

5.9.3 Pedestrian Button Signs

Pedestrian push buttons are normally installed to ensure that the full sequence of pedestrian indications is displayed upon demand. These push buttons should be located near each end of crosswalks where actuation is required. A mounting height of 3.5 feet above the sidewalk has been found to be best adapted to general usage. Signs should be mounted above the push button assembly, explaining their purpose and use. Where two crosswalks, oriented in different directions, end at or near the same location, the positioning of pedestrian push buttons should clearly indicate which crosswalk signal is actuated by each push button. Special signs to clarify the application of pedestrian push buttons are prescribed in ***MUTCD*** and as shown in ***Figure 5-5***.

Figure 5-5 Special Signs for Pedestrian Buttons



Additional push-button assemblies may be required on islands or medians where a pedestrian might become stranded. Other forms of pedestrian detection and confirmation are covered in Chapter 4.

5.10 SIGNS AND MARKINGS FOR MEDIANS AND DIVIDED HIGHWAYS

Intersections with divided highways, pose difficulties for drivers, especially for older drivers at night. Typical intersections with a set of reflectorized signs and markings are shown for an unsignalized intersection in ***Figure 5-6***. A distinction is made between divided highways with medians less than 30 feet in width and those with a median wider than 30 feet. ***Figure 5-6*** shows the case of an intersection with median greater than 30 feet. For intersections with narrow medians the ONE-WAY signs and the traffic control signs on the median itself are eliminated. ***Design Standards, Index 17349*** presents the standard treatment for "T" intersections on divided highways.

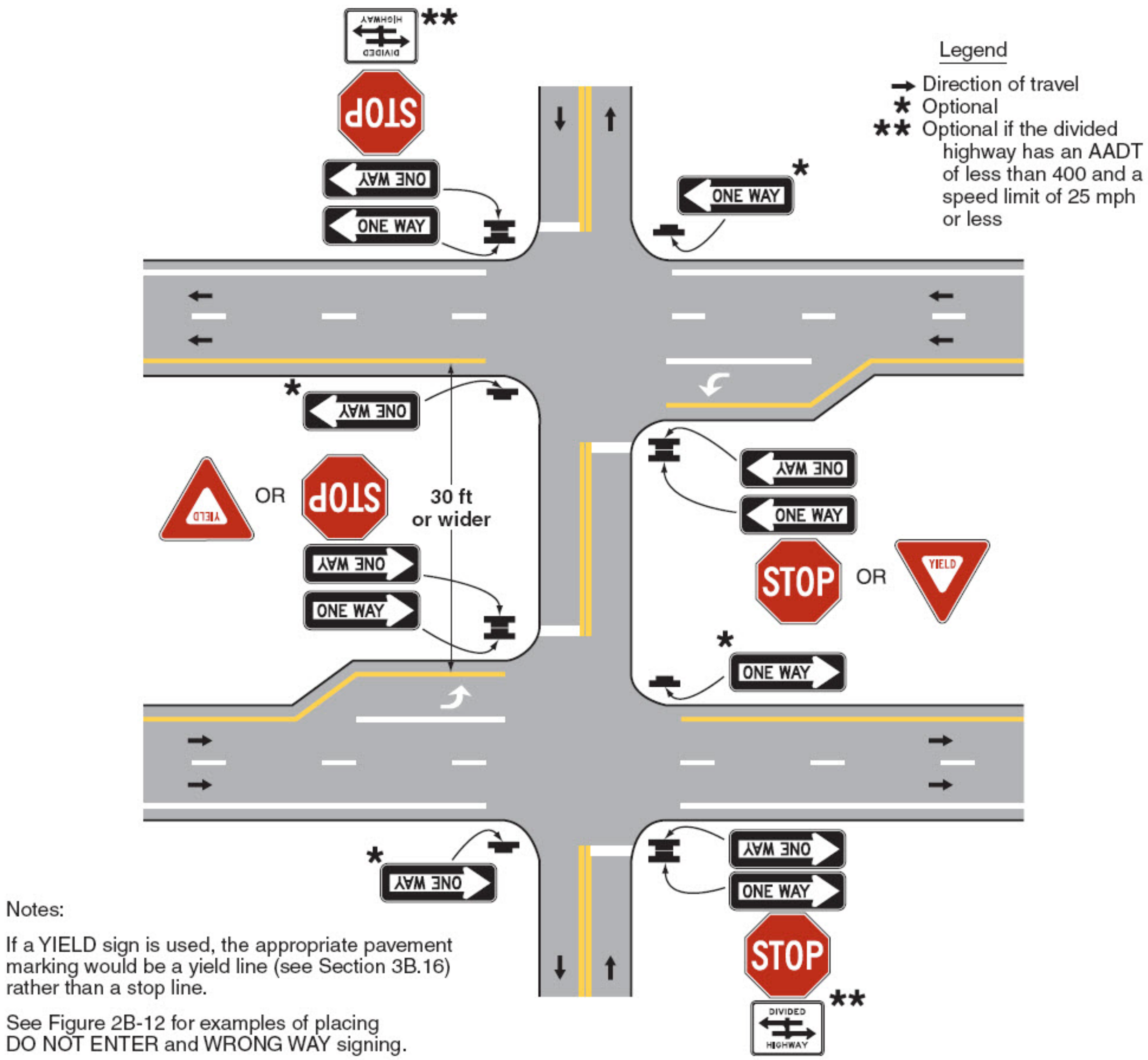


Figure 5-6 Signing for Wide Medians

5.11 ROUNDABOUT SIGNS AND MARKINGS

Due to substantial safety, operational, and capacity characteristics, roundabouts are an equivalent or better intersection configuration for any new roadway or reconstruction project. Additional discussion regarding signs and markings for roundabouts are included in ***NCHRP 672***.

5.11.1 Advance Warning

Advance warning signs may be required along the approach to a roundabout. All signs must comply with the ***MUTCD***. The warning sign is optional if the speed on the approach road does not exceed the roundabout design speed by more than 10 mph. If the speed limit on the approach is greater than 10 mph above the roundabout design speed, the advance warning sign is recommended along with an advisory speed sign plate (as prescribed in the ***MUTCD***). A supplementary panel with the crossing street name can be helpful for navigation. Yield Ahead signs and pavement marking can also be used in advance of a roundabout.

Figure 5-7 Advance Warning Signs for Roundabouts (W2-6)

For multi-lane roundabouts, advance route signage as well as lane assignment signs and markings are recommended. "Fish Hook" arrows and corresponding signs are especially useful to guide the motorist through a roundabout.

5.11.2 Entry Treatment

Figure 5-8 illustrates the entry treatment recommended for roundabouts by the ***MUTCD***. A YIELD sign is required at each entrance to the roundabout. Note that a yield line marking is required to supplement the Yield Sign. Yield lines shall consist of a row of solid white isosceles triangles pointing toward approaching vehicles extending across approach lanes to indicate the point at which the yield is intended or required to be made. The individual triangles comprising the yield line should have a base of 12 to 24 inches wide and a height equal to 1.5 times the base. The space between the triangles should be 3 to 12 inches. There should be no painted lines across the exits from a roundabout. YIELD legend markings are optional.

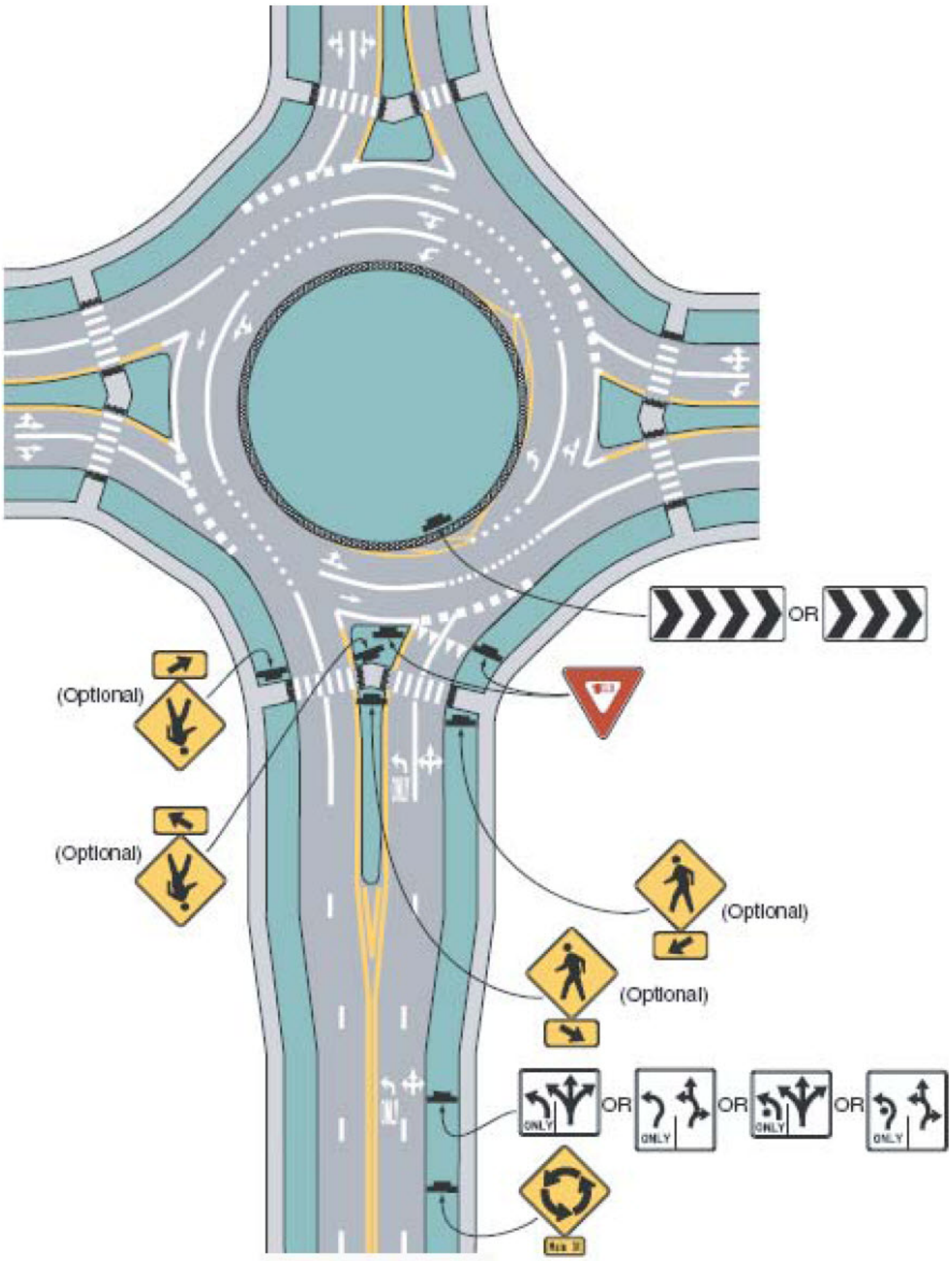


Figure 5-8 **Roundabout Signing and Marking Requirements *(MUTCD)***

5.11.3 Circulatory Roadway

Standard signing inside the inscribed circle includes chevron signs across from each approach. Lane lines are useful for maintaining lane discipline within the circle.

5.11.4 Accommodation of Pedestrians and Bicyclists

Although separate paths for bicyclists may be used at high volume roundabouts, no special markings or lanes are used. Approach roadways that have bicycle lanes should end at a ramp to the sidewalk for less experienced bicyclists, or allow merging with through traffic to command the lane through the circulatory roadway.

The provisions for pedestrians do not change the geometric design requirements from treatments required for other intersections. Large roundabouts can result in greater walking distances, but special crossing facilities are not necessary. Well designed splitter islands that can harbor anticipated pedestrians will allow them to cross only one direction of traffic at a time. Crossing lines should not be painted at the entrances and exits at the roundabouts. Crossings should be situated approximately 20 feet from the yield line as illustrated in ***Figure 5-8***.

Where consideration must be given to priority crossings for pedestrians, additional signs and markings will need to be supported by appropriate analysis and study of specific candidate sites.

5.12 SPECIALIZED SIGNING AND MARKING DETAILS

The focus of this chapter has been on at-grade intersections that include primarily motorized vehicles on approach roadways. Several unique situations may arise at intersections. For example, there are specialized situations where cross traffic includes only bicycles, pedestrians or golf carts, usually at mid-block crossings.

Innovative signing and marking treatments may be required to solve specific problems. It is not the intent of this document to discourage innovation. It is important, however, that all innovative measures be designed within the framework of existing standards and criteria. In particular, the designer should refer to ***Design Standards, Index 17346*** for details pertaining to markings, in general, and to ***Design Standards, Index 17355*** for special sign details.

Chapter 6

Objects and Amenities

Tables

Figures

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6 OBJECTS AND AMENITIES

All intersection features not covered in ***Chapters 3, 4 and 5*** are grouped together in this chapter under the general heading of *objects and amenities*. The topics include landscaping, lighting, utilities, on-street parking and public transit facilities. Each of these topics will be covered separately.

6.1 SIGNIFICANT REFERENCES

The following references contain information that applies to objects and amenities at intersections:

AASHTO Roadway Lighting Design Guide;

NCHRP 152 "Warrants for Highway Lighting";

FDOT Highway Landscape Guide;

Utility Accommodations Manual (UAM);

A Policy on Geometric Design of Highways and Streets (AASHTO Green Book);

Florida Manual of Uniform Minimum Standards for Design, Construction and Maintenance for Streets and Highways (Florida Greenbook);

FDOT *Plans Preparation Manual*;

***Transit Capacity and Quality of Service Manual* (TRB Publication)**

TCRP Report 19: *Guidelines for the Location and Design of Bus* Stops*.*

6.2 LANDSCAPING

"Landscape" or "Landscaping" means any vegetation, mulches, irrigation systems, and any site amenities, such as, street furniture, specialty paving, tree grates, walls, planters, fountains, fences, and lighting (excluding public utility street and area lighting), as defined in ***Rule Chapter 14-40, Florida Administrative Code, Highway Beautification and Landscape Management***. Landscaping may be constructed as a stand-alone project or as a component of a roadway project or Community Feature.

6.2.1 Placement of Plants at Intersections

It is important that plants not be allowed to infringe on the motorist's ability to see other vehicles and objects that could present a hazard or complicate the driving task. In the case of roundabouts, landscaping can be used to block sight distance through the roundabout to increase the visibility of the roundabout itself, reduce headlight glare, and to avoid the appearance of a continuous corridor. The geometric criteria that affect driver visibility are covered in Chapter 3. The concepts of horizontal clearance and clear zone are covered in ***Section 3.10.2,*** and ***Section 3.14*** covers sight distance requirements. Additional requirements are provided in ***PPM Volume I, Chapter 2*** and FDOT ***Design Standards, Index 546 Sight Distance at Intersections***. All landscaping must comply with these requirements.

6.3 LIGHTING

Proper lighting at intersections can provide important safety benefits in addition to a general road-user amenity. Highway safety improvement projects evaluated by FHWA suggest that intersection illumination is one of the most effective treatments for reducing serious crashes. Proper lighting reduces improper and wrong-way maneuvers by increasing visibility of roadway geometry, signing and pavement markings. The principal design considerations include selection of an appropriate level of illumination and the design details of the physical installation.

According to the 2005 AASHTO *Roadway Lighting Design Guide*, lighting may be provided for all major arterials in urbanized areas and for locations or sections of streets and highways where the ratio of night to day crash rates is higher than the statewide average for similar locations and a study indicates that lighting would significantly reduce the nighttime crash rate. Additional warranting conditions are presented in **NCHRP Report 152 *Warrants for Highway Lighting***. Lighting is required at all roundabouts.

Also, lighting may be considered at locations where severe or unusual weather or atmospheric conditions exist. In other situations, local governmental policies determine the type of lighting features to consider.

For lighting on roadways not on the SHS, the ***Florida Greenbook*** prescribes warranting conditions for the provision of roadway lighting.

The AASHTO *Roadway Lighting Design Guide* permits either the illuminance technique or the luminance technique to be used in the design of highway lighting. The luminance technique requires a more complex design process and knowledge of the reflective characteristics of the pavement surface used. These reflective characteristics change as the pavement ages and with variations in weather conditions. The Department has elected to use the illuminance technique for lighting design.

The lighting design criteria for the Department are contained in the ***PPM,*** Volume I, Chapter 2. The design values for light levels given by the AASHTO *Roadway Lighting Design Guide* are maintained values. The light levels given in the ***PPM*** have been adjusted and are listed as average initial values.

6.3.1 Location of Lighting Poles

The primary sources of standards and guidelines for the location of lighting poles are found in the ***Design Standards*** as follows:

Index 17500 shows details for conventional lighting poles and bases.

Index 17502 shows details for high mast lighting poles and bases.

Index 17515 shows the requirements that apply specifically to aluminum poles.

The physical roadside conditions may restrict the placement of lighting poles. Safety considerations for lighting pole locations are as follows:

1. Pole locations must meet horizontal clearance requirements. See Chapter 3, Section 3.10.2 and ***PPM,*** Volume I, Chapter 2.
2. Pole locations should consider the hazards in servicing the lighting equipment.
3. Poles should be placed to minimize interference with the driver’s view of sign legends and the luminaire brightness should not seriously detract from sign legibility at night.
4. Poles should not be placed where overhead signs will cast distracting shadows on the roadway surface at night.
5. Poles should never be placed on the traffic side of guardrail or any natural or manmade deflecting barrier.
6. Where poles are located in exposed areas, they shall be designed to have a suitable breakaway or yielding feature. Poles made of lightweight metals have some value in reducing vehicle damage from secondary impact, particularly when the collision speed is relatively low and the falling pole makes a secondary contact with the vehicle.
7. Poles shall not be located in the median except in conjunction with barriers that are justified for other reasons.
8. The installed lighting system should have a pleasant daytime appearance and reflect aesthetic considerations.

High Mast Lighting

High mast lighting is a type of lighting with groups of luminaires mounted on free standing poles or towers, at mounting heights varying from approximately 80 feet to 180 feet, that enables the development of a highly uniform light distribution.

This type of lighting provides excellent uniformity of illumination and reduces glare with a substantially smaller number of pole locations, especially at interchanges and other complex road areas. It also provides a contribution to safety and aesthetics by reducing the number of poles needed and through locating poles out of the recovery area adjacent to the driving lanes. Their remote location eliminates the need for maintenance vehicles obstructing traffic on the roadway or the requirement for maintenance personnel to be near the high-speed traffic lanes.

The most common type of luminaire used in high mast lighting is the area type with either a symmetric or asymmetric distribution. Due to the lack of satisfactory experience in designing high mast installations to the luminance system, use of the luminance system is not encouraged when designing a high mast installation. In addition to the level of light on the roadway, the designer must also consider objectionable spill light and discomfort glare beyond the right of way and the visibility of vertical surfaces on the roadway system, i.e., guardrail, bridge columns, abutments, drainage, headwalls, etc.

High mast lighting is not generally used except on limited access facilities and interchanges. High mast lighting is therefore only applicable to intersections involving cross roads and ramps.

6.4 UTILITIES

The primary concerns in the design and location of utility installations are the protection of the highway facility and the safety of the highway user while giving full consideration to sound engineering principles and economic factors. The ***PPM,*** Volume I, Chapter 5 Utilities, provides specific direction on utility considerations that must be addressed in the design of state highway facilities. The ***Utility Accommodation Manual (UAM)*** establishes requirements for the accommodation of new and existing utilities along, across, on, or under transportation facilities within FDOT rights of way.

The ***UAM*** was developed to support the FDOT in its responsibility for *coordinating the planning of a safe, viable, and balanced State transportation system serving all regions of the state, and to assure the compatibility of all components, including multi-modal facilities,* given under ***Florida Statutes, Section 334.044(1)***. In addition, ***Florida Statutes, Section 337.401(2)***, provides that *no utility shall be installed, located, or relocated unless authorized by a written permit issued by the FDOT.* A utility permit application must be submitted by the Utility Agency/Owner (UAO). An engineer or contractor may prepare and process a permit application for a utility owner, but shall not be identified as the permittee.

The ***UAM*** draws upon many resources as guidelines to establish standards for utility work or placement and reimbursement cost within the rights of way. When a FDOT standard is found to be more stringent, the FDOT standard shall apply.

The key topics related to intersection design covered by the ***UAM*** include:

1. Utility permit requirements, application and processing;
2. Accommodation standards for all types of utility installations;
3. Maintenance of the utility, vegetation, etc.;
4. Maintenance of traffic;
5. Location criteria;
6. Utility surveys;
7. Special requirements and exceptions and
8. References.

The ***UAM*** covers a wide variety of subjects, many of which are not related to intersection design and operation. A summary of the important requirements that do relate specifically to intersections is presented as follows:

1. Any installation that requires a structural modification to an FDOT facility must be signed and sealed.
2. Crossings under existing pavement will usually be made without cutting the pavement.
3. Designs for utility attachments shall be in compliance with all applicable federal, state and local regulations, rules, and codes.
4. All materials and methods to be used for utility conduit, pipe coatings and concrete repairs shall be approved by the FDOT's State Materials Office.
5. Where it is necessary to place temporary supports for aerial crossings that will interfere with traffic, careful planning of work with regard to the safety of vehicular traffic is mandatory.
6. In any analysis of a request for open cutting or trenching, primary considerations will be given to the safety and convenience of the public. The applicant shall provide written justification for approval of open cutting.
7. When a permit for utility installation, adjustment or maintenance activity is required, a proposed traffic control plan shall be submitted with the permit application for approval.
8. All new utility/light pole installations shall comply with the ***UAM*** horizontal clearance and clear zone criteria. On construction projects where the permittee cannot meet these requirements, the designer shall determine what additional safety requirements are needed.
9. Aerial crossings are permitted and will have a minimum of 18 ft. vertical clearance over the roadway.
10. Underground crossings require a minimum vertical clearance of 36 inches below top of pavement and 30 inches below existing unpaved ground line, including ditch grade.
11. Devices such as signal strain poles, fire hydrants (where practical), down guys, telephone load pedestals and other items whose construction and size would cause extensive damage to a vehicle if struck are to be located according to the same horizontal clearance standards applied to utility poles.
12. Manholes and valve boxes shall be outside the travel way and bike lanes, to the greatest extent practical. The manhole ring, cover and pad must support the traffic for the area where it is being constructed and shall be set flush with the finished grade.
13. Exceptions are required when any one of the following criteria or policies is not complied with: vertical clearance, horizontal clearance, limited access right of way use, control zone use, clear zone or MSE walls.

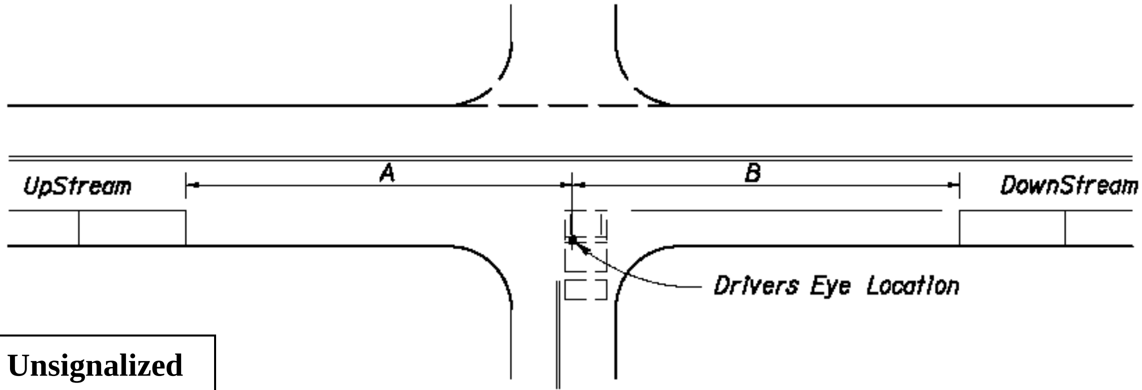
6.5 ON-STREET PARKING

Arterial highways are designed specifically for the movement of vehicles. This implies that parking on urban arterial streets and rural arterial highway sections should be prohibited whenever possible. However, some arterials must also serve the adjacent land development (such as in urban environments) and this function may result in the need for parking on the street.

6.5.1 Relationship to Intersections

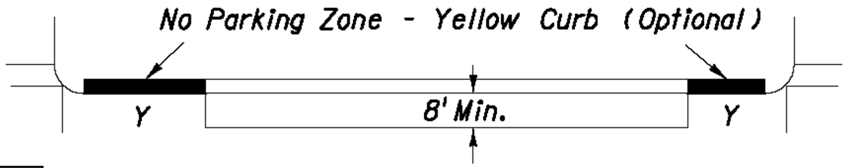
Areas adjacent to intersections are of particular interest with regard to congestion. Where parking is permitted too close to intersections the result is blocked sight distances and poor visibility of vehicles and pedestrians. Vehicles parked close to intersections often block lanes that could be used by drivers to bypass left-turning vehicles. The amount of clearance required is a function of the speed in the adjacent traveled lanes. The minimum clearances for parking at both signalized and unsignalized intersections, as specified in ***Design Standards, Index 17346*** are illustrated in ***Figure 6-1***.

Parking prohibitions can be justified under three conditions: statutory, capacity effect and hazard. A national consensus is available in the statutory warrants of the Uniform Vehicle Code and Model Traffic Ordinances which authorizes full-time prohibitions on both sides of roadways not exceeding 20 feet wide, and on one side of those not over 30 feet wide. ***Florida Statutes*** specifically prohibit parking within 20 feet of a crosswalk at an intersection, and within 30 feet upon the approach to any flashing signal, stop sign, or traffic control signal. Where bicycle routes have been designated where parking is present, the bike lane should be placed between the parking lane and the travel lane and have a minimum width of 5 feet.



a. Unsignalized Intersections

SPEED MPH	UP STREAM (A)	DOWN STREAM (B)	
		2 LANE	4 LANE
0-30	85'	60'	45'
35	100'	70'	50'



b. Signalized Intersections

SPEED LIMIT MPH	SIGNALIZED INTERSECTIONS
0 - 30	30
35	50

DISTANCE FROM CURB RADIUS (Y)

Figure 6-1 Minimum Parking Clearances for Signalized and Unsignalized Intersections (*Design Standards*, *Index 17346)*

6.6 PUBLIC TRANSIT FACILITIES

The ***Florida Greenbook*** defines public transit as passenger transportation service, local or regional in nature that is available to any person. It operates on established schedules along designated routes or lines with specific stops and is designed to move relatively large numbers of people at one time. Public transit includes bus, light rail and rapid transit. The FHWA recommends that bus service on arterial roadways should be limited to regional or long-distance routes.

6.6.1 Intersection Capacity Considerations

The vehicle-carrying capacity of the through lanes is decreased when the transit vehicle and other traffic use the same lanes. A bus stopping for passenger loading, not only blocks traffic in that lane but also affects operation in all lanes. Therefore, coordination with the local transit agency is important to determine the best locations for transit routes and stops. Bus stops on arterial roads, where necessary, should be designed with bus bays to remove the bus from through traffic.

On roadways where buses turn right at a corner, the front end may overhang into side street opposing lanes. The latter should be a consideration when placing stop lines on cross streets.

6.6.2 Bus Stop Locations

The general location of bus stops is largely dictated by patronage and by intersecting bus routes or transfer points. Furthermore, the specific location is influenced by the design characteristics and operational considerations of the highway.

Except where cross streets are widely spaced, bus stops are usually located in the immediate vicinity of intersections, facilitating crossing streets by patrons without the need of mid-block crosswalks.

ADA Requirements for Bus Stops

1. Firm, stable surface
2. Minimum clear length of 96 inches
3. Minimum clear width of 60 inches (measured parallel to the vehicle roadway)
4. Connected to streets, sidewalks or pedestrian paths
5. The slope of the pad parallel to the roadway shall be the same as the roadway.
6. Maximum slope: (2%) perpendicular to the roadway

Bus stops at intersections may be placed on the near (approach) side or the far (exit) side. In some cases it may be advantageous to place bus stops at mid-block locations instead of intersections.

There are many considerations that affect the choice of bus stop placement. A thorough discussion of this subject may be found in TCRP Report 19: *Guidelines for the Location and Design of Bus Stops*. ***Table 6-1***, reproduced from the TCRP report, summarizes the relative advantages and disadvantages of each placement choice. If there is an established bus route, bus bays shall always be placed on the near side on the approach to one-lane roundabouts to minimize the probability of vehicle queues spilling back into the circulatory roadway. Bus stops located on the far side of a roundabout intersection need to have pullouts or be further downstream than the splitter island.

Where practicable, bus stops on cross streets should be located and arranged so that transferring riders are not required to cross the arterial highway, regardless of the direction they wish to travel.

Bus stops may also be located on frontage roads of arterial streets, where buses leave and return to the artery by special openings in the outer separation in advance of and beyond an intersection. Thus, buses stop in positions well removed from the through lanes. Right-turning traffic to and from the arterial street may also be accommodated at these special openings, thereby reducing conflicts at the intersection.

According to the ***Florida Greenbook***, bus benches should be set back at least 10 feet from the travel lane in curbed sections and outside the clear zone in non-curbed sections. Any bus benches or transit shelters located adjacent to a sidewalk within the right of way of any road on the State Highway or County Road System shall be located so as to leave at least 36 inches clearance, measured in a direction perpendicular to the centerline of the road. A separate bench pad or sidewalk flare is required by ADA, for pedestrians and persons in wheelchairs. If roadways are to be widened or resurfaced, consultation with the local transit agency is recommended to determine the need to remove, relocate, or replace transit stops, benches, shelters and other amenities.

Table 6-1 Comparative Analysis of Bus Stop Locations (*TCRP Report 19*)

	Advantages	Disadvantages
Far-Side Stop	• Minimizes conflicts between right turning vehicles and buses • Provides additional right turn capacity by making curb lane available for traffic • Minimizes sight distance problems on approaches to intersection • Encourages pedestrians to cross behind the bus • Creates shorter deceleration distances for buses since the bus can use the intersection to decelerate • Results in bus drivers being able to take advantage of the gaps in traffic flow that are created at signalized intersections	• May result in the intersections being blocked during peak periods by stopping buses • May obscure sight distance for crossing vehicles • May increase sight distance problems for crossing pedestrians • Can cause a bus to stop far side after stopping for a red light, which interferes with both bus operations and all other traffic • May increase number of rear-end accidents since drivers do not expect buses to stop again after stopping at a red light • Could result in traffic queued into intersection when a bus is stopped in travel lane
Near-Side Stop	• Minimizes interferences when traffic is heavy on the far side of the intersection • Allows passengers to access buses closest to crosswalk • Results in the width of the intersection being available for the driver to pull away from curb • Eliminates the potential of double stopping • Allows passengers to board and alight while the bus is stopped at a red light • Provides driver with the opportunity to look for oncoming traffic, including other buses with potential passengers	• Increases conflicts with right-turning vehicles • May result in stopped buses obscuring curbside traffic control devices and crossing pedestrians • May cause sight distance to be obscured for cross vehicles stopped to the right of the bus • May block the through lane during a peak period with queuing buses • Increases sight distance problems for crossing pedestrians
Mid-block Stop	• Minimizes sight distance problems for vehicles and pedestrians • May result in passenger waiting areas experiencing less pedestrian congestion	• Requires additional distance for no-parking restrictions • Encourages patrons to cross street at midblock (jaywalking) • Increases walking distance for patrons crossing at intersections

Bus Bays on Arterial Roadways

Providing bus bays for buses to pull out of travel lanes and clear the lanes for through traffic can considerably reduce the interference between buses and other traffic. Where necessary, bus bays are strongly encouraged on arterial highways. To be effective, bus bays should incorporate the following features:

1. A deceleration lane and entry taper to permit easy entrance to the loading area: The taper should be designed at an angle flat enough to encourage the bus operator to pull completely clear of the through lane. A taper of 5:1, longitudinal to transverse, is a desirable minimum. When the bus stop is on the far side of an intersection, the intersection area may be used as part of the entry area to the stop. When nearside stops are planned, signal prioritization for the bus to reenter traffic should be considered. Nearside stops should be designed so that vehicles making a right turn will not queue in the bus turnout.

2. A standing space or loading area sufficiently long to accommodate the maximum number of vehicles expected to occupy the space at one time: The loading area should provide about 50 feet of length for each bus. Articulated buses will need more space. The additional length requirement can be determined through consultation with the transit agency. The width should be at least 12 feet. Narrower lanes will cause the bus to partially obstruct the adjacent lane.”

3. A merging lane to enable easy reentry into the through-traffic lanes. This lane may be sharper than the deceleration taper, but, preferably, should not be sharper than 3:1. If the bus bay is on the near side of an intersection, the width of cross street is usually great enough to provide the necessary merging space.

The total length of a bus bay of minimal design for a two-bus loading area and width of 11 feet should be approximately 180 feet for a midblock location, 140 feet for a near-side location and 125 feet for a far-side location. These dimensions should be increased by 12 to 15 feet for a loading area of 12 feet in width. Bus stops on routes that have articulated buses will also need additional length considerations.

An alternative treatment for bus stops strictly in urban areas is called a bus bulb. A bus bulb is an arrangement by which the curb and sidewalk is extended toward the center of the road for a bus stop; typically replacing roadway area that would otherwise be part of a parking lane. With bus bulbs, a bus can stay in its traffic lane to discharge and pick up passengers, instead of having to pull over to the curb. Transit drivers may be more likely to use bus bulbs than bus bays. Other considerations for using bus bulbs is that they allow for more parking, take less right-of-way, and provide a safe, level platform for passengers boarding and alighting. Like turnouts, bus bulbs should have adequate passenger storage areas.

For additional guidance on the location and placement of bus facilities, see FDOT’s ***Accessing Transit Handbook***.

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Chapter 7

Roundabouts

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7 ROUNDABOUTS

7.1 INTRODUCTION

Due to substantial safety, operational, and capacity characteristics, roundabouts are an equivalent or better intersection configuration for any new roadway or reconstruction project. Roundabouts shall be evaluated on new construction, reconstruction and safety improvement projects, as well as anytime there are proposed changes in intersection control that will be more restrictive than the existing conditions. General design standards, criteria, etc. shall be in conformance with the ***NCHRP Report 672, Roundabouts: An Informational Guide*** (TRB, 2010), which is established as FDOT policy by adoption. This chapter establishes additional requirements that may differ from, or supplement direction provided in the ***NCHRP Report***.

On State roads or at intersections with State roads, only modern roundabouts as defined in ***NCHRP 672*** will be allowed. This shall not prohibit local agencies from constructing other types of circular intersections on roads and streets wholly within their jurisdiction. Roundabouts may be constructed at any intersection that is currently, or planned to be, under two-way or multi-way stop control, or signalized. Intersections may have three, four or more approaches and can have either single-lane or two-lane approaches. Roundabouts may be especially useful as a practical solution to skewed intersections. At this time, roundabouts shall have a maximum of two lanes in the circulatory roadway; although spiral or turbo designs (varying number of lanes) are acceptable and three-lane roundabouts may be considered in the future. The decision to construct a roundabout must be included in all intersection analyses.

An initial screening shall be conducted at the Preliminary Engineering (PD&E) stage of every project to determine which of the intersections within the project limits will be constructed as roundabouts. The initial screening for applicability of roundabouts is based on traffic volumes. Roundabouts shall be considered the preferred option at intersections where the total projected (design year) entering traffic volumes are up to 25,000 vehicles per day (vpd) for a single-lane, and up to 45,000 vpd for a two lane roundabout. Minor street volumes should not be less than 1,500 vpd nor more than 12,000 vpd for a single-lane, and between 1,500 and 27,000 vpd for a two lane roundabout. Intersections exhibiting volumes outside these limits may be candidates for roundabouts subject to an operational analysis. The following checklist presents a summary of the recommended considerations that could preclude construction of a roundabout.

Roundabout Screening Checklist

- ☐ Traffic Operations
- ☐ Bike/Pedestrian need
- ☐ Local Jurisdictional Policies
- ☐ Available Right-of-Way
- ☐ Existing Structures
- ☐ Business Access
- ☐ Environmental Constraints
- ☐ Historical Sites
- ☐ <30,000 vehicles entering (1-lane)
- ☐ <50,000 vehicles entering (2-lane)
- ☐ >90% of all traffic entering on main street
- ☐ Vehicle Mix

Double-roundabout interchanges (with a roundabout at each direction's ramp terminal) can effectively replace traffic signals at diamond interchanges, and will handle peak-hour entering volumes of up to 4,500 vehicles. In addition to reducing control delay, double-roundabout interchanges typically have reduced cross sections on the connecting roadway due to the absence of auxiliary lanes, thus a narrower bridge is required for the overpassing street.

If an intersection passes the initial screening, an individual isolated operational analysis of each roundabout shall be performed in compliance with the ***Highway Capacity Manual*** (TRB, 2010). A series of roundabouts shall be analyzed using corridor simulation. Operational analyses will require the development of future year turning movement volumes, and a comparison of the roundabout to a conventional intersection. A corridor simulation shall analyze the affects to coordinated signal progression (if present) in the area. Operational analyses shall compare levels of service and delay for a roundabout versus a conventional intersection. If a roundabout exhibits equal or better operating and safety characteristics than a conventional intersection, a roundabout must be completely advanced through the public involvement (CAP) stage.

Once an intersection or series of intersections within a project has passed the operational screening, a safety and construction benefit/cost analysis shall be undertaken. The benefit/cost analysis shall consider the benefits of crash and delay reduction of a roundabout in comparison to the construction and right-of-way costs that *exceed* those of a proposed conventional intersection. The analysis shall also account for the savings in right-of-way costs over a conventional intersection due to the elimination of additional turn and/or auxiliary lanes and their transitions. If the benefit/cost ratio of a roundabout is greater than that for a conventional intersection, a roundabout becomes the preferred alternative

7.2 ROUNDABOUT CONSIDERATIONS

Where roundabouts are used, they shall be made readily apparent to motorists. The geometric features of the roundabout should be the chief means of making roundabouts visible, rather than through the primary use of signs and markings. Pedestrian features (where applicable) shall be plainly obvious through the use of crosswalks and supplementary signs, and shall include splitter islands sufficiently sized to harbor anticipated pedestrian group sizes (6' minimum at the crosswalk). Night time illumination is mandatory to increase the visibility of the roundabout and improve sight distance during dark hours. Roundabouts shall not include stop signs or signalization and parking is prohibited within the circulatory roadway. Driveways shall not be allowed within the circulatory roadway unless sufficient traffic volumes exist to consider the driveway as an additional leg. Driveways also should not be permitted in the functional vicinity of the splitter island.

Right-of-way requirements for roundabouts are generally greater at the intersection itself, but this may be offset by the reduction in right-of-way on the approach roadways due to the absence of left turn, right turn or other auxiliary lanes and their transitions. Roundabouts can reduce the number of lanes required on connecting roadway segments because traffic passing through a roundabout is effectively metered, leading to more random release and higher efficiency in lane usage following departure. Access management and safety is facilitated by providing means for U-turns within the circulatory roadway, allowing more restrictive median treatments between roundabouts.

Roundabouts by nature encourage lower speeds on the approach to, and within the circulatory roadway, thereby enhancing safety characteristics. The numbers of vehicles that are required to come to a complete stop at a roundabout are significantly less than at a conventional intersection, thereby reducing delay. Because entering vehicles are required to yield to vehicles within the circulatory roadway, sight distance is critical to entering vehicles, while approaching vehicles should not be given the appearance of a linear path. The use of mounded central islands with landscaping or community features is encouraged to increase the visibility of the roundabout itself and further reduce approach speeds.

In complex settings such as skewed or other than four approaches, advance guide signs are encouraged to assist in navigation through a roundabout. T and Y intersections may also benefit from advance guide signs, especially where the major through movement may involve a change in direction. Diagrammatic signs are particularly useful for this purpose, and proportionate lane widths for departing roadways are also helpful.

Bicyclists shall be offered two paths upon approach to the roundabout. An existing or planned bicycle lane on an approach roadway shall end in advance of the roundabout with a ramp providing access to the sidewalk. At the end of the bicycle lane, the cyclist may either "command the lane" and pass through the circulatory roadway, or divert onto the sidewalk and cross at pedestrian crossings. No bicycle lane markings shall be placed within the circulatory roadway. This treatment shall also be applied in the absence of bicycle lanes.

In general, roundabouts should be considered under any of the following conditions:

1. New construction
2. Reconstruction
3. Alternative to adding lanes
4. History of fatal and injury crashes[a]
5. History of left-turn and right-angle crashes
6. History of pedestrian or bicycle crashes
7. One or two approach lanes per leg (not including turn lanes)
8. Three approach lanes (with right-turn bypasses)
9. Total vehicles entering a single-lane circle between 15,000 – 30,000 daily[b]
10. Total vehicles entering a dual-lane circle between 30,000 – 50,000 daily[c]
11. Three or more than four approach roadways
12. Skewed or unusual approach geometry
13. Interchange ramp termini
14. All-way stop control warranted
15. Signalization warranted
16. Locations where signalization has been requested
17. Limited sight distance
18. Heavy minor street delay at 2-way stop controlled intersection
19. Access Management
20. Context Sensitive Solution (CSS) or other special treatment areas

Contraindicating factors: The term “contraindication” is defined in the *Webster’s Dictionary* as “something (as a symptom or condition) that makes a particular treatment or procedure inadvisable”. A contraindicating factor for selecting a roundabout as an intersection control device would be any condition that might reduce the effectiveness of a roundabout. Keep in mind that almost all intersections have conditions that could be considered as contraindicating factors for any intersection control device, including signals. World-wide experience has shown that there are a few conditions under which roundabouts may not perform well enough to be considered as the most appropriate form of control. These factors must be examined carefully as a part of the justification process. Although these factors may not preclude the choice of a roundabout, they would indicate that there is a potential problem and that mitigation efforts should be detailed in the study process.

[a] Crashes of types subject to correction through the installation of a roundabout.
[b] At locations exceeding 25,000 a detailed capacity analysis is required.
[c] At locations exceeding 45,000 a detailed capacity analysis is required.

A number of contraindicating factors are listed below. Certain other physical or geometric complications may make it impossible or uneconomical to construct a roundabout. These could include right of way limitations, utility conflicts, drainage problems, etc; proximity of generators of significant traffic that might have difficulty negotiating the roundabout. For example, a fire station right at the intersection, or an institution that serves blind people might be considered a contraindication; Proximity of other traffic control devices that would require preemption, such as railroad tracks, drawbridges, etc.

1. Proximity of bottlenecks that would routinely back up traffic into the roundabout, such as overcapacity signals, freeway entrance ramps, etc; Problems of grades or unfavorable topography that may limit visibility or complicate construction;

2. Intersections of a major arterial and a minor arterial or local road where an unacceptable delay to the major road is created. Roundabouts delay and deflect all traffic entering the intersection which could cause excessive delay to the major arterial;

3. Heavy pedestrian movements that would have trouble crossing the road because of high traffic volumes. This indication would also include special need pedestrian areas (areas with a large number of children, elderly people, etc.);

4. Isolated intersections located within a coordinated signal network. In these situations, the level of service of the arterial might be better with a signalized intersection incorporated into the system;

5. Roadways with reversible lanes for morning and afternoon peak periods;

6. Routes where large combination vehicles or over-dimensional vehicles will frequently use the intersection and insufficient space is available;

7. Locations where vehicles exiting the roundabout would be interrupted by downstream traffic control that could create queues backing up into the roundabout; and,

8. Areas with a large number of cyclists. Although roundabouts readily accommodate the bicyclist, areas with a large number of bicyclists and insufficient crossing opportunities (due to high traffic volumes) would need additional review and evaluation.

9. Proximity of historical sites, socially significant trees, etc.

The existence of one or more of these conditions does not necessarily preclude the installation of a roundabout. However, the presence of any contraindication suggests that special attention should be given to the design and operation to ensure that problems do not arise.

7.3 PLANNING

As stated previously, roundabouts are the preferred intersection configuration for any new roadway or reconstruction project. Roundabouts need not be considered for pavement only projects (POP), unless there are other indications of need.

The following policy-level issues should be considered in the decision to construct a roundabout:

- Capacity and Level of Service
- Bike/Pedestrian treatment
- Local Jurisdictional Policies
- Community Acceptance
- Business Access
- Environmental Constraints
- Historical Sites
- Growth Potential
- Bus Stops

7.4 OPERATIONAL ANALYSIS

Operational analyses for roundabouts are presented in Chapter 4 of ***NCHRP 672***, and in Chapter 21 of the ***Highway Capacity Manual***, ***TRB***, 2010. Level of Service for roundabouts is measured in control delay consistent with other unsignalized intersections. Operational analyses of roundabouts require the collection or projection of peak period turning movement volumes. Other required data for the analysis includes the following:

1. The number and configuration of lanes on each approach;
2. Either of the following:
    a. Demand volume for each entering vehicular movement and each pedestrian crossing movement during the peak 15 minutes, or
    b. Demand volume for each entering vehicular movement and each pedestrian crossing movement during the peak hour, and a peak hour factor for the hour;
3. Percentage of heavy vehicles;
4. Volume distribution across lanes for multilane entries; and
5. Length of analysis period, generally a peak 15 minute period within the peak hour.

Numerous software packages implementing the procedures contained in the ***Highway Capacity Manual*** have been developed, and are recommended for operational analyses of isolated roundabouts. In cases where a second roundabout, multi-way stop, or signalized intersection falls within ½ mile of the subject roundabout, a systems level operational analysis will be required using software specifically designed for roundabouts in a system.

7.5 SAFETY

Roundabouts are recommended as a significant safety countermeasure by the FHWA, which presents the following statistics:

- By converting from a two-way stop control mechanism to a roundabout, a location can experience an 82 percent reduction in severe (injury/fatal) crashes and a 44 percent reduction in overall crashes.
- By converting from a signalized intersection to a roundabout, a location can experience a 78 percent reduction in severe (injury/fatal) crashes and a 48 percent reduction in overall crashes.

Specific guidance from FHWA indicates that roundabouts should be considered as an alternative for intersections on federally funded highway projects that involve new construction or reconstruction. Roundabouts should also be considered when rehabilitating existing intersections that have been identified as needing major safety or operational improvements. Roundabouts have also proven to be effective at freeway interchange ramp terminals and to reduce speeds at rural high-speed intersections on the approach to congested areas.

7.6 GEOMETRIC DESIGN

Roundabouts should be designed to reduce traffic speeds; high-speed entries are discouraged. In Florida, high-speed entries are defined as follows:

Single-lane roundabouts – Speeds in excess of 25 mph

Multi-lane roundabouts – Speeds in excess of 30 mph

Roundabouts should be designed for operating speeds between 20 and 25 mph. The approaches to the roundabout must be carefully designed to provide enough deflection to cause motorists to reduce speeds prior to entry. Single-lane roundabouts should be designed to accommodate the design vehicle, while two-lane roundabouts should accommodate the design vehicle and a passenger car astride in traversing the roundabout. Vehicle paths must be checked through the roundabout to ensure no encroachment between the vehicles, while minimizing the fastest-path speeds. At a minimum, roundabouts should accommodate school buses, moving vans, garbage trucks, fire trucks, and other emergency vehicles.

The circulatory roadway of a roundabout should be designed with the minimum practical diameter in order to keep circulating speeds low. The exception is when a roundabout is designed for future expansion as traffic volumes increase. The expansion of a roundabout can be accommodated by reducing the central island diameter (widening to the inside) or increasing the overall diameter of the circulatory roadway (widening to the outside). While conventional intersections are commonly designed for ultimate demand (usually a 20-year projection), roundabouts should be implemented in stages, by expanding only when traffic volumes demand.

The use of a mounded central island and landscaping is encouraged to increase the visibility of the roundabout itself to approaching motorists, limit the “straight-through” appearance of the intersection and reduce night time headlight glare.

7.7 TRAFFIC CONTROL DEVICES

Traffic control devices are important to proper operations in a roundabout. Yield signs (R1-2) are required on the right side of all entries, and shall be double-posted on multi-lane approaches. Pavement markings, lane use signs, and destination signs are used to guide motorists in lane selection and conveying the allowable movements. Destination signs posted in advance of the roundabout should be diagrammatic, and provide general information regarding the connecting routes (road numbers or names). On the immediate approach to the intersection, lane-use signs (fish-hook arrows) and corresponding pavement markings assist in lane selection by entering vehicles. Yield lines shall be used on every entry to roundabouts located adjacent to Yield signs. Lane lines within the circulatory roadway should be as recommended in ***NCHRP 672***, with white dotted lines crossing the entries. No lines shall cross the exit lanes of roundabouts. Roundabout directional arrow signs (R6-4 series) should be used in the central island at the far side of approach roadways. Pedestrian crossing signs shall be installed at all crosswalks.

7.8 LIGHTING

All roundabouts on the state highway system shall have night time illumination. The minimum standard of lighting is 1.5 foot-candles on the roadway surface within the circulatory roadway and for a minimum of 250 feet in advance of the yield lines on the approach roadways.

7.9 LANDSCAPING

Landscaping is encouraged at all roundabouts to assist in raising the visibility of the roundabout itself, as well as, channeling pedestrians away from the circulatory roadway. Care must be taken in landscaping layout to avoid intruding into sight distance triangles. Standard Index 546, sheet 7 of 7 provides measures of sight distance requirements. FDOT’s preference is for trees and turf in the central island, with low-growing shrubbery between the sidewalks and the circulatory roadway.

7.10 CONSTRUCTION AND MAINTENANCE

Roundabouts are most easily constructed when an intersection can be closed to traffic. Closing an intersection may be unreasonable, so roundabouts are generally constructed in two phases. The first phase would be construction of the additional pavement required to the outside limits of the roundabout and approach roadways. A central island can then be laid out using barricades or traffic barrels, with yield signs and lane use control signage to establish the roundabout movements. With the outside limits constructed, traffic is moved away from the center of the intersection while the central island and splitter islands are constructed. Because of limited space, single-lane roundabouts may require the use of temporary pavement outside the limits of the ultimate roundabout and temporary barrier wall to provide safe clearance for workers.

Maintenance agreements should be sought with local agencies wherever possible, and especially in cases of community features. As mentioned previously, landscaping is useful in discouraging pedestrian traffic from crossing the central island, and adds safety benefits for motor vehicles.

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Appendix A

Intersection Plateauing Example

Intersection profiles are commonly designed so that the profile of the primary roadway takes precedence over the cross street by carrying the cross slope of the primary roadway through the intersection and transitioning the profile and edge of pavements of the cross street to match the edge of pavement of the primary roadway. However, because the primary roadway is often a multilane highway, the profile of the cross street through the intersection can be less than desirable for the following reasons:

1. The sight distance on the cross street may be restricted to the point where the lane lines are not visible across the intersection.
2. The cross street profile often has abrupt changes that are particularly undesirable for signalized intersections where the cross street traffic may enter the intersection without stopping.

The conditions sited above can be more pronounced when both roadways are multilane highways. The cross street may have similar traffic demands as the primary roadway and, thus, deserves “equal treatment” with respect to the profile. In this case, a technique known as intersection plateauing is applied in which the roadway profiles and cross slopes for all approaches to the intersection are transitioned. Intersection plateauing should be considered at signalized intersections and is particularly applicable at intersections between two multilane roadways. The following steps summarize the procedure for developing profiles of a plateau intersection:

1. Set the Approach Profile Grade Lines and Resulting Cross Slopes
2. Determine the Station and Elevation of the Intersecting Roadways
3. Locate the Edge-of-Pavement Transitions
4. Calculate Elevations along the Lane Lines
5. Develop the Curb Return Profiles for Returns without Islands
6. Develop the Curb Return Profiles for Returns with Islands
7. Check the Right Turn Lane Relative Gradient and Cross Slope Break-Over

Example: Given an intersection between two multilane roadways, SR 70 and CR 70, develop profiles using the plateauing procedure. SR 70 is a six lane facility and CR 70 is a four lane facility as shown in ***Exhibit 1***.

<u>STEP 1: Set the Approach Profile Grade Lines and Resulting Cross Slopes</u>

According to the 2004 AASHTO Manual, page 582, “The alignment and grades are subject to greater constraints at or near intersections than on the open road. At or near intersections, the combination of horizontal and vertical alignment should provide traffic lanes that are clearly visible to drivers at all times, clearly understandable for any desired direction of travel, free from potential for conflicts to appear suddenly, and consistent in design with portions of the highway just traveled." In order to provide a comfortable ride and sight distance through the intersection, it is desirable to provide the flattest grades practical while providing at least minimum gradients to ensure adequate longitudinal drainage in curbed sections.

Minimum gutter grades should not be flatter than 0.3% according to the PPM and the Drainage Manual. However, AASHTO allows a minimum of 0.2%. If a grade flatter than 0.3% is used the designer should check with the District Drainage Engineer as a design variation may be required.

In this example, the project area has level terrain. The intersection will be designed with 0.5% grades on all four approaches with the high point located in the middle of the intersection. This will provide good sight distance across the intersection and accommodate at least AASHTO minimum gutter grades at all the returns.

STEP 2: Determine the Station and Elevation of the Intersecting Roadways

The approach grades for both roadways should extend far enough back to include the edge of pavement transitions. The edge of pavement should transition to a plateau at or before the beginning of the curb return. Referring to ***Exhibit 1***, Curb Return No. 1 in the Northwest quadrant, and Curb Return No. 3 in the Southeast quadrant will be referenced.

West Approach: Begin Station of RETURN 1 = 119+07.02 CL-SR70.

Transition Length = 114.00' (see step 3 for transition length transitions)

Begin 0.5% approach grade before STA.117+93.02 CL-SR70.

East Approach: End Station of RETURN 3 = 121+33.48 CL-SR70

Transition Length = 114.00'

Begin 0.5% approach grade after STA. 122+47.48 CL-SR70.

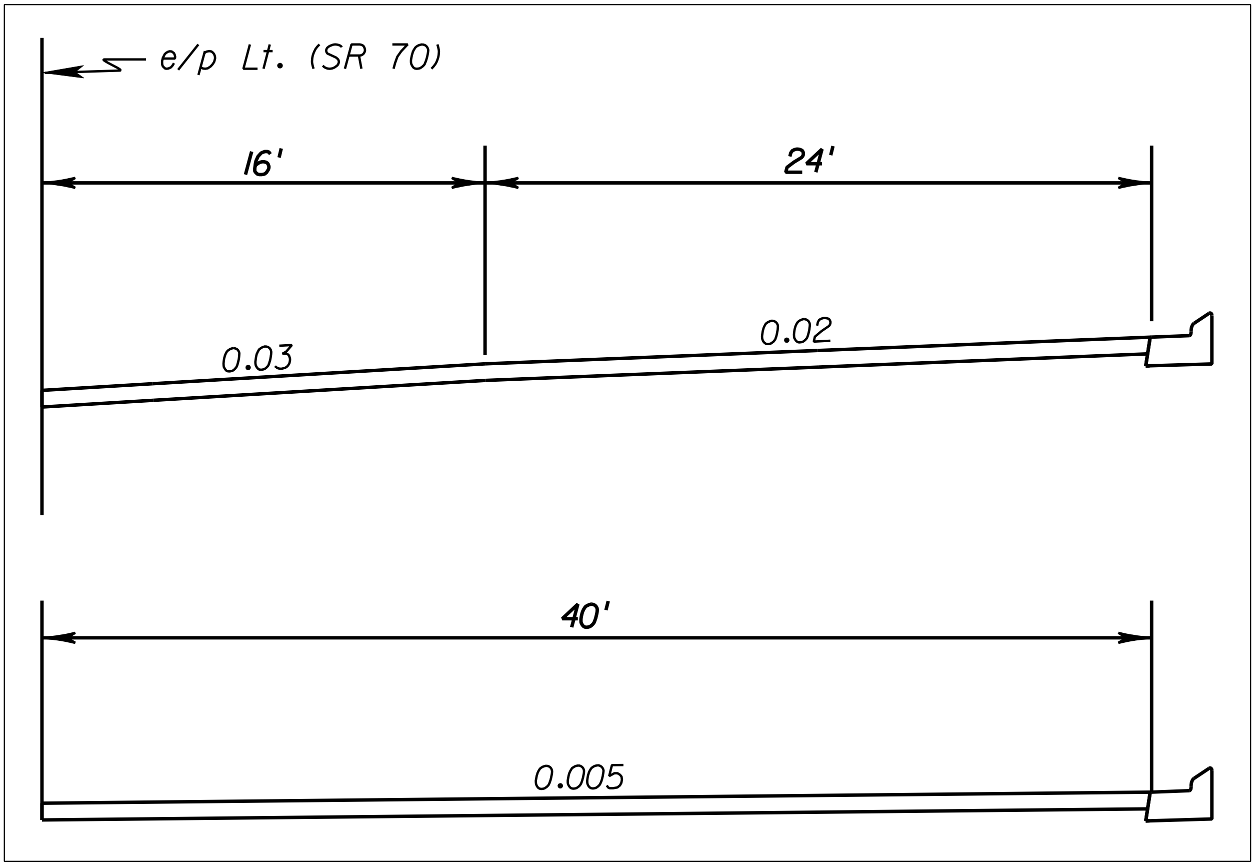
The PGL of SR 70, as shown in ***Exhibit 2***, can be adjusted vertically to best fit the project profile, accommodate the plateau transitions and maintain the roadway base clearance above the high water elevation. PIs on the profile grade line (PGL) of SR 70 are set at STA. 117+00 and STA 124+25. This sets the intersecting profile grade elevation of 35.16 at STA. 120+18.97. The side street (CR 70) profile is then set to this elevation as shown in ***Exhibit 2***.

When setting the profile grade lines through an intersection it can be the case that constraints exist in the vicinity of the intersection. It is always the case that the PGLs are developed to best accommodate back-of-sidewalk or property line profiles and drainage. Therefore, the development of the PGLs may require several iterations to develop the best fit for the particular project.

STEP 3: Locate the Edge-of-Pavement Transitions

Referring to the SR 70 sections in ***Figure A-1***, calculate the change in elevation of the edge of pavement between the normal crown (NC) section and the elevated crown (EC) section that represents the plateau at the intersection.

Figure A-1



NC elevation = (24'x -0.02'/ft.) + [(12'+4')x-0.03] = -0.96'

EC elevation = (36'+ 4') x -0.005 = -0.2'

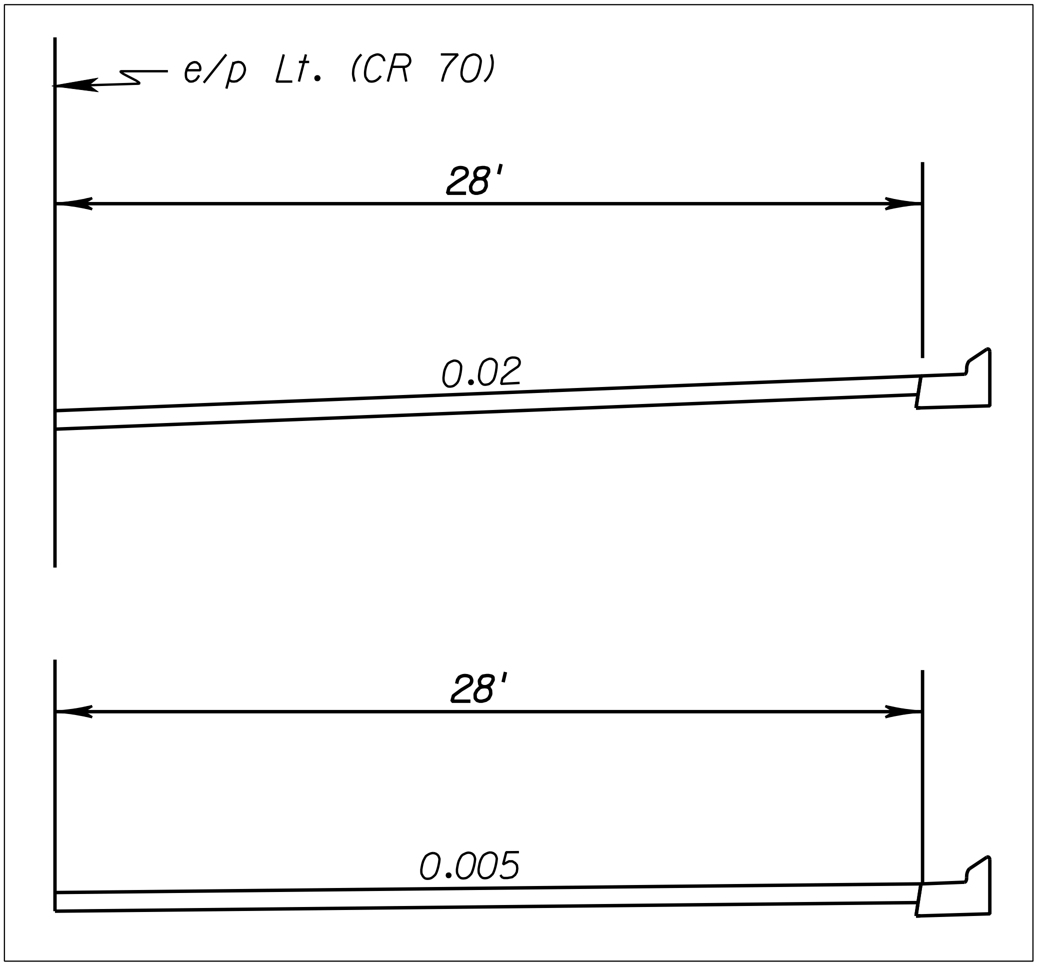
EC - NC = -0.2' - (-0.96') = 0.76'

From Table 2.9.4 in the ***PPM*** the transition slope rate for urban highways with a design speed of 45 mph is 1:150. Therefore, the transition length for the e/p-SR 70 will be:

0.76' x 150 = 114.00'

Referring to the CR 70 sections in **Figure A-2**, calculate the change in elevation of the edge of pavement between the normal crown section and the elevated crown section representing the plateau at the intersection.

Figure A-2



NC elevation = (24' + 4') x -0.02 = -0.56'

EC elevation = (24' + 4')*-0.005 = -0.14'

EC - NC = -0.14' - (-0.56') = 0.42'

0.42' x 150 = 63.0'

West Approach: end the transition at STA. 119+07.02. Therefore;

Begin Transition = 119+07.02 - 114.00' = STA. 117+93.02.

The outside lane will transition from a 3.0% cross slope at STA. 117+93.02 to a 0.5% cross slope at STA 119+07.02.

East Approach: begin the transition at STA. 121+33.48. Therefore;

End Transition = 121+33.48 + 114.00' = STA. 122+47.48.

The outside lane will transition from a 0.5% cross slope at STA. 121+33.48 to a 3.0% cross slope at STA. 122+47.48.

South Approach: end the transition at STA. 549+66.51. Therefore;

Begin Transition = 549+66.51 - 63’ = STA. 549+03.51.

The outside lane will transition from a 2.0% cross slope at STA. 549+03.51 to a 0.5% cross slope at 549+66.51.

North Approach: begin the transition at STA. 552+15.46. Therefore;

End Transition = 552+15.46 + 63 = STA. 552+78.46.

The outside lane will transition from a 0.5% cross slope at STA. 552+15.46 to a 2.0% cross slope at STA. 552+78.46.

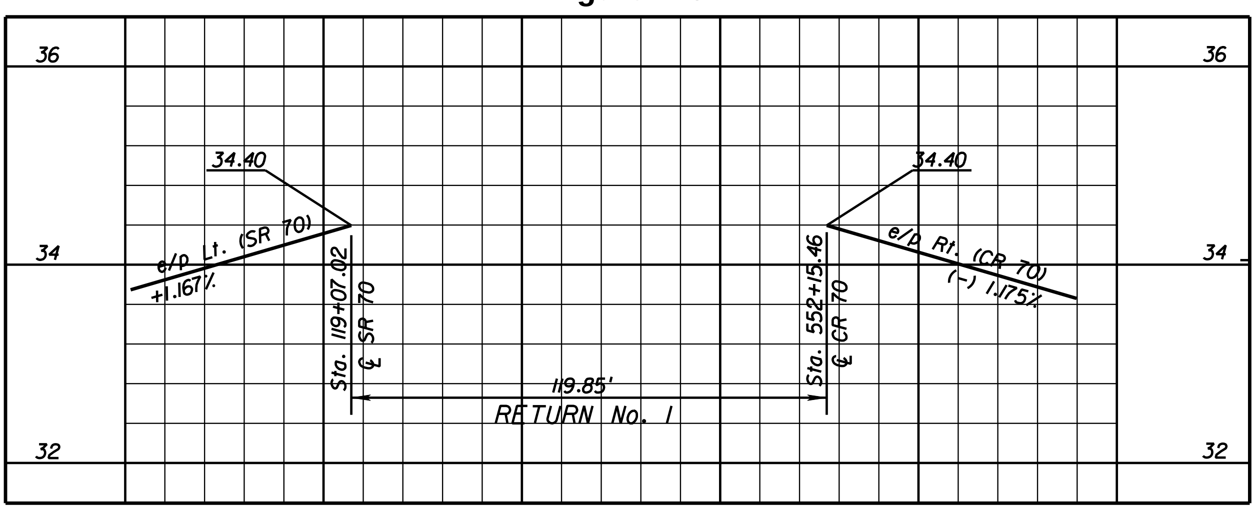
STEP 4: Calculate Elevations along the Lane Lines

Certain elevations within the intersection should be labeled on the Intersection Detail Sheet. These elevations are generally at the intersection of the lane lines, including the PGL lines, at the PCs and PTs of curb returns, and at certain locations on the raised islands. These elevations can be obtained from chains and profiles stored in GEOPAK or hand calculated. ***Exhibit 1*** shows the locations where elevations and station/elevation labels should be placed.

STEP 5: Develop the Curb Return Profiles for Returns without Islands

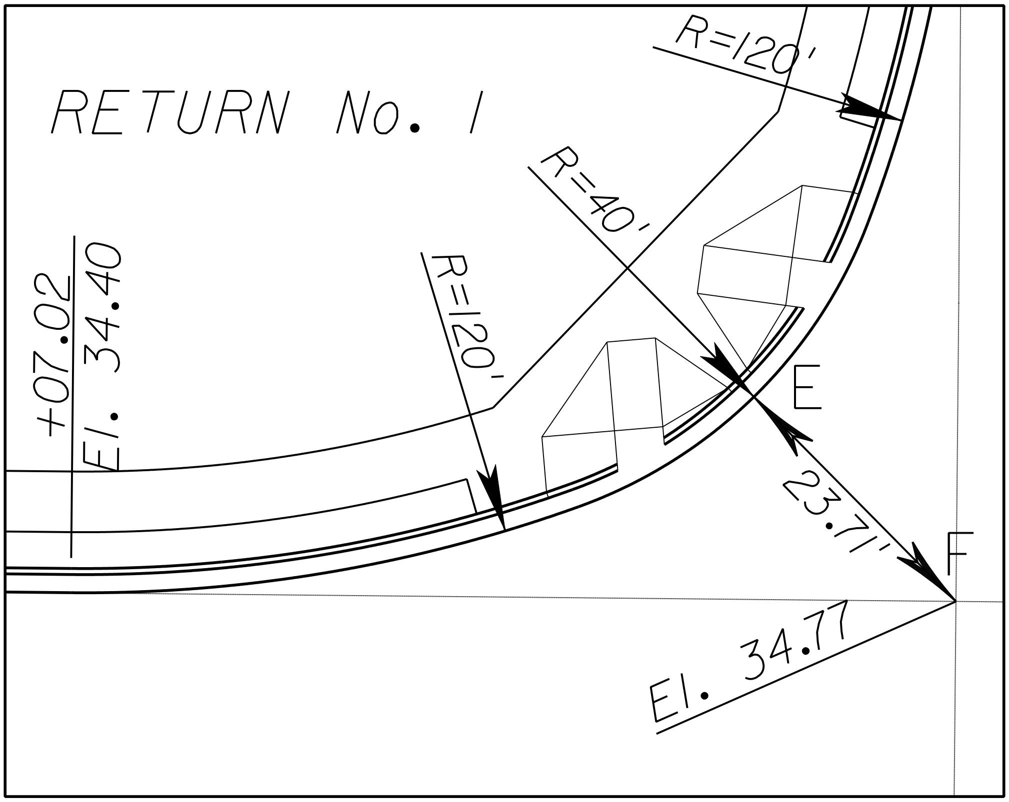
Referring to RETURN No. 1, measure the length of the return from PC to PT. Since this return is a three-centered curve it is the distance from the PC of the first curve to the PT of the third curve. Plot the return dimension on profile view and label the PC and PT stations. Next, plot the edge of pavement approach grades (see ***Exhibit 2***) and the elevations as shown in ***Figure A-3***.

Figure A-3



In order to accommodate ADA curb ramps and drainage it is desirable to set the high point of profile in the center of the return. This can be accomplished as long by measuring the distance from point 'E' to point 'F' as shown in ***Figure A-4***.

Figure A-4

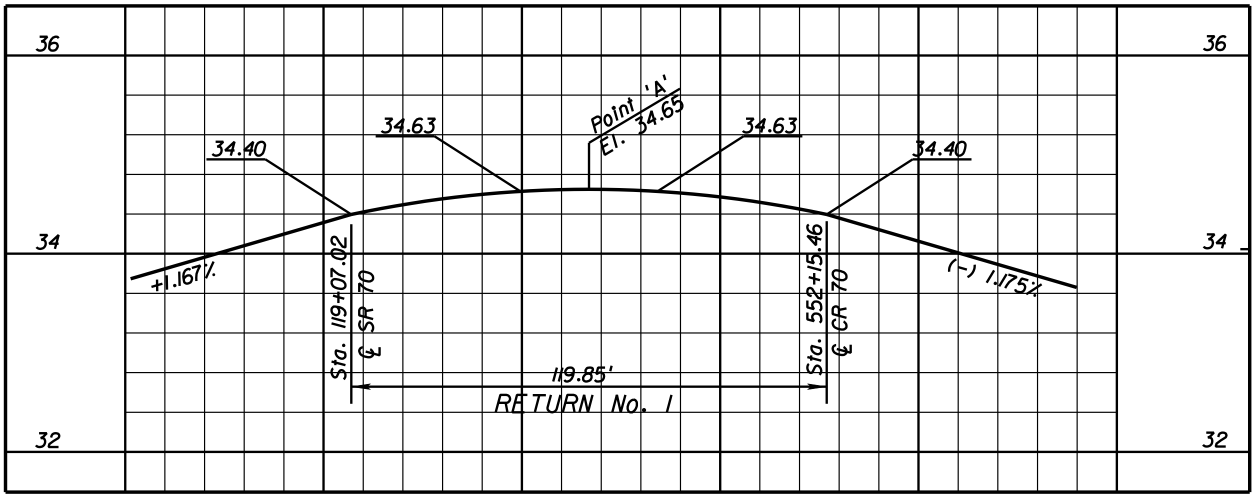


Calculate the elevation of point 'E' by setting a similar grade, in this case 0.5%, along vector 'E-F':

El. 'E' = 34.77' – (23.71' * 0.005) = 34.65'

Plot a point in the center of the return profile at elevation 34.65' and fit a spline curve through the end points and the center point as shown in ***Figure A-5***. Check that the return has positive drainage in both directions. Repeat STEP 5 for RETURN No. 3.

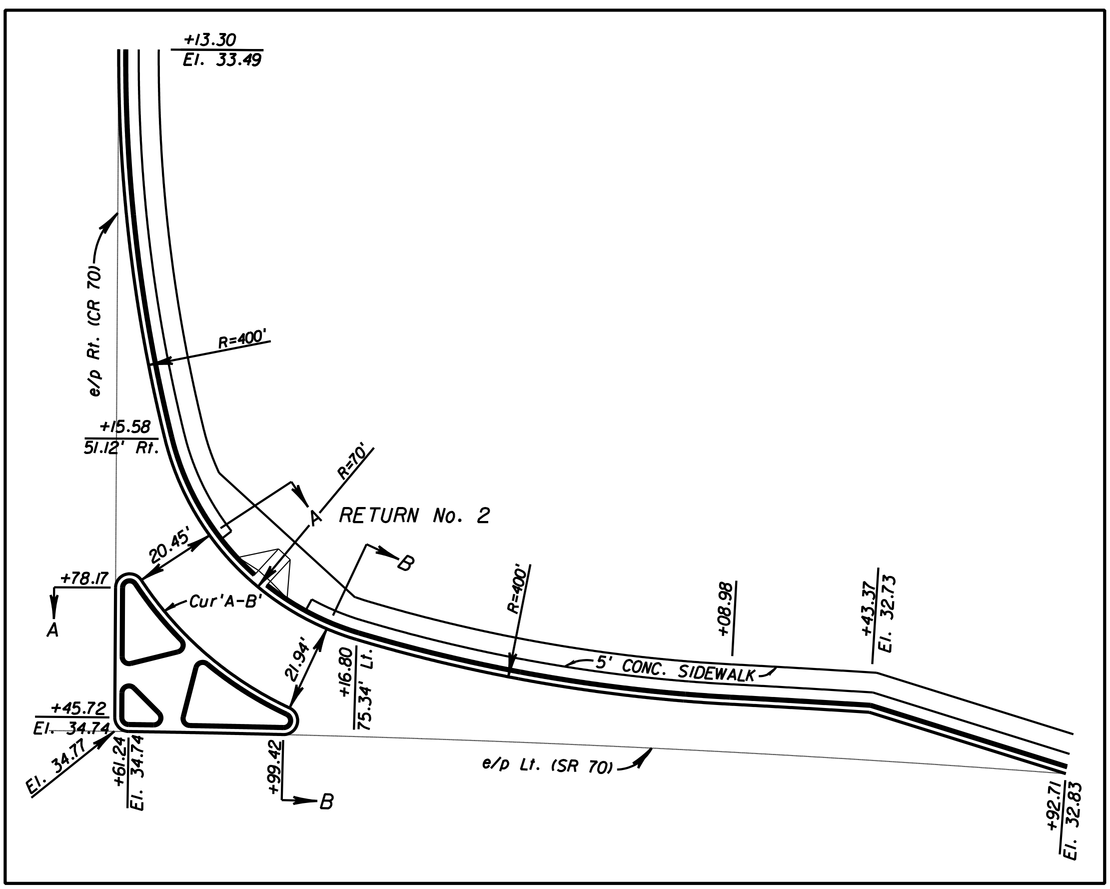
Figure A-5



STEP 6: Develop the Curb Return Profiles for Returns With Islands

With the introduction of a right turn lane and a raised island, special profiles and sections will be required to accurately depict the design. Consider RETURN No. 2 as shown in ***Figure A-6***:

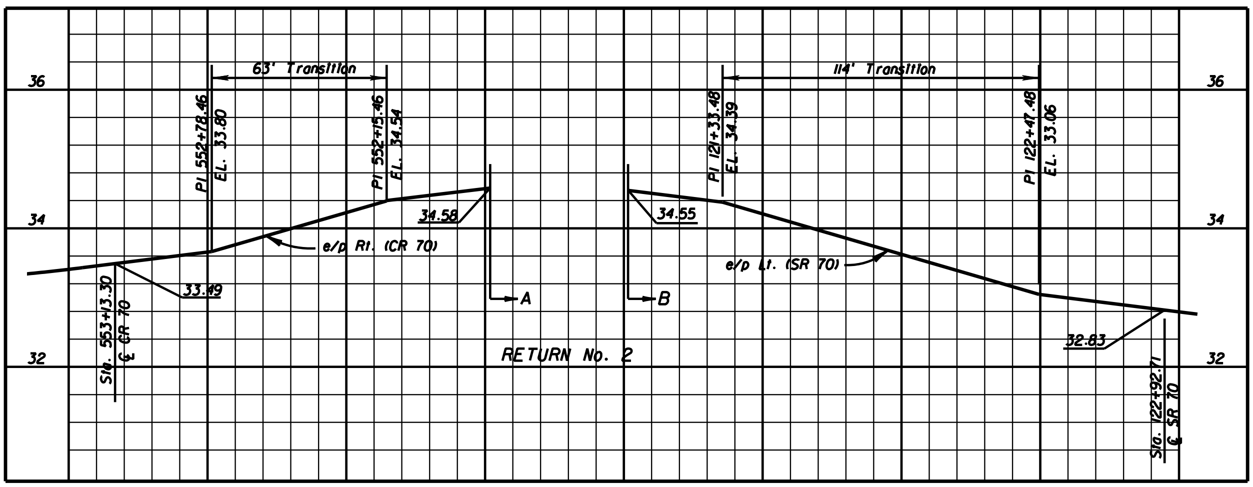
Figure A-6



The elevations along the two sides of the raised island adjacent to the thru-lanes are set by the edge of pavement profiles. First, sections ‘A-A’ and ‘B-B’ will be developed along with the profile of Curve ‘A-B’. The grades on the island proper will be checked for compliance to ADA requirements. The edge of pavement return profile will then be developed.

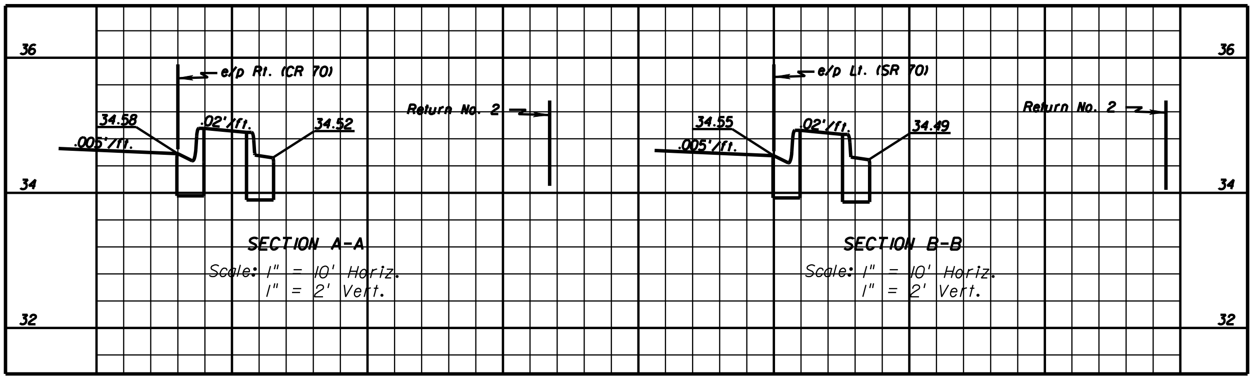
Plot the edge of pavement profiles for the two approaches from beyond the limits of the return to sections ‘A-A’ and ‘B-B’. Label the known stations and elevations as shown in ***Figure A-7***.

Figure A-7



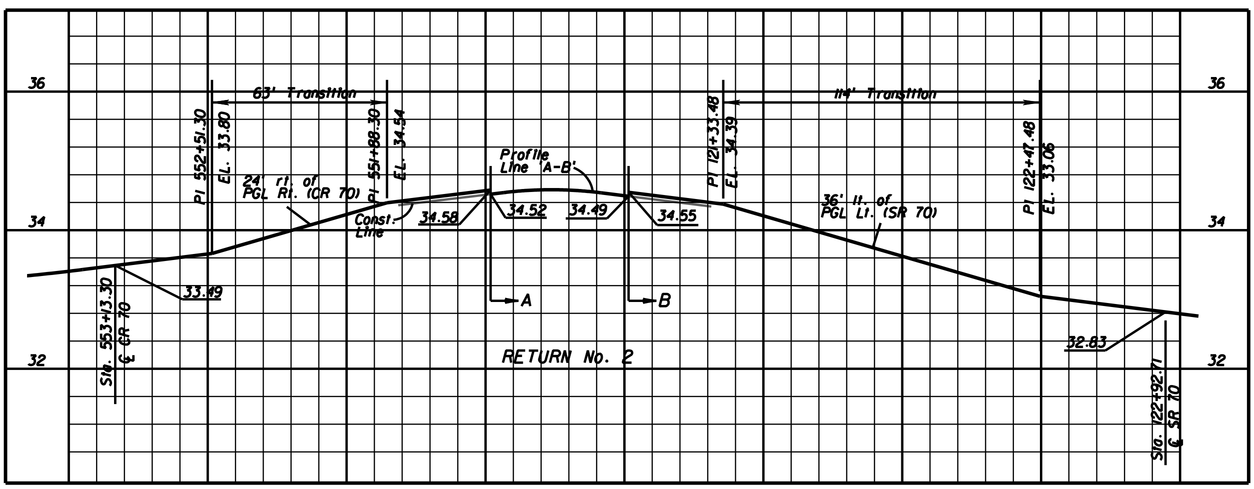
Begin plotting sections ‘A-A’ and ‘B-B’ by plotting the cross slope of the through lane and the cross section of the raised island. In this example the cross slope is 0.005’/ft. and the curb is type ‘F’. The section will go through the radius point of the island nose; therefore, the distance across the island will be twice the radius, or 7.0 ft. The surface of the island between the curbs should be set on a slope. In this case, the surface between the curbs measures 3.0 ft. Using a 0.02’/ft. slope, the edge of pavement along Curve ‘A-B’ at Sections ‘A-A’ and ‘B-B’ will be 0.06 ft. below the edge of pavement of the through lane. The width of the right turn lane at sections ‘A-A’ and ‘B-B’ are known from the plan view, but the elevation of the return at these points is not known yet. Plot the known information on an exaggerated scale as shown in ***Figure A-8***.

Figure A-8



Plot the elevation on Cur 'A-B' at section 'A-A', and plot the elevation on Cur 'A-B' at section 'B-B'. The profile of Cur 'A-B' will be drawn between these two elevations and generally match the profile of the return. Using tangents sloped at .005'/ft. (shown here as construction lines) place a spline curve representing the profile of Cur 'A-B' as shown in ***Figure A-9***.

Figure A-9



In this example, the middle of the intersection is the high point; therefore, the profile of Cur 'A-B' and the return profile are crowned so that water drains away from the middle of the intersection to each approach. If one or both of the roadway profiles are carried through the intersection on a tangent, this would make it necessary to slope the entire return to one side or the other. The objective is to not have a low point in the vicinity of the cross walks and curb ramps so that drainage structures do not interfere with these features and water does not puddle where pedestrians may be walking.

Certain controls will be applied in developing the return profile from the beginning of the return on CR 70 to the end of the right turn taper on SR 70. The right turn lane is a 'turning roadway' and should be designed as such within certain conditions and constraints. These conditions include the necessity to yield to pedestrians and conflicting traffic movements, and the need to not exceed maximum ADA slopes and minimum gutter grades. According to the 2001 AASHTO Manual, page 639, “In intersection design, the free flow of turning roadways is often of limited radii and length. When speed is not affected by other vehicles, drivers on turning roadways anticipate the sharp curves and accept operation with higher side friction than they accept on open highway curves of the same radii. When other traffic is present, drivers will travel more slowly on turning roadways than on open highway curves of the same radii because they must diverge from and merge with through traffic. Therefore, in designing for safe operation, periods of light traffic volumes and corresponding speeds will control.” According to the AASHTO superelevation tables, a superelevation rate of 5.0% will accommodate a 70.0’ radius curve at about 15 mph to 20 mph. Therefore, using maximum superelevation will introduce higher side friction at higher speeds as explained above. In addition, the maximum cross slope of the right turn lane at the crosswalk is a 1:20 slope to accommodate ADA requirements. Therefore, start by considering a 0.05’/ft. cross slope.

The width of the turn lane is approximately 21’.

21’ x .05 = 1.05’

Therefore, start by drawing the return profile between section ‘A-A’ and ‘B-B’ about 1.05’ below profile line ‘A-B’. The elevation of the return at section ‘A-A’ will be:

34.52’ - 1.05’ = 33.47’

Next, check the gutter grade from the end of the return on CR 70 to section ‘A-A’. The length of the gutter section is 135.13’. The grade will be:

(33.47’ - 33.49’)/135.13’ = -0.00015

Since this grade is well below the minimum grade, the gutter grade will control. In order to gain as much superelevation as possible, we will deviate from state standards and use a 0.2% gutter grade.

Calculate the elevation of the return at section ‘A-A’ using a 0.2% gutter grade:

33.49’ + (135.47’ x 0.002) = 33.76’

Calculate the cross slope at section ‘A-A’:

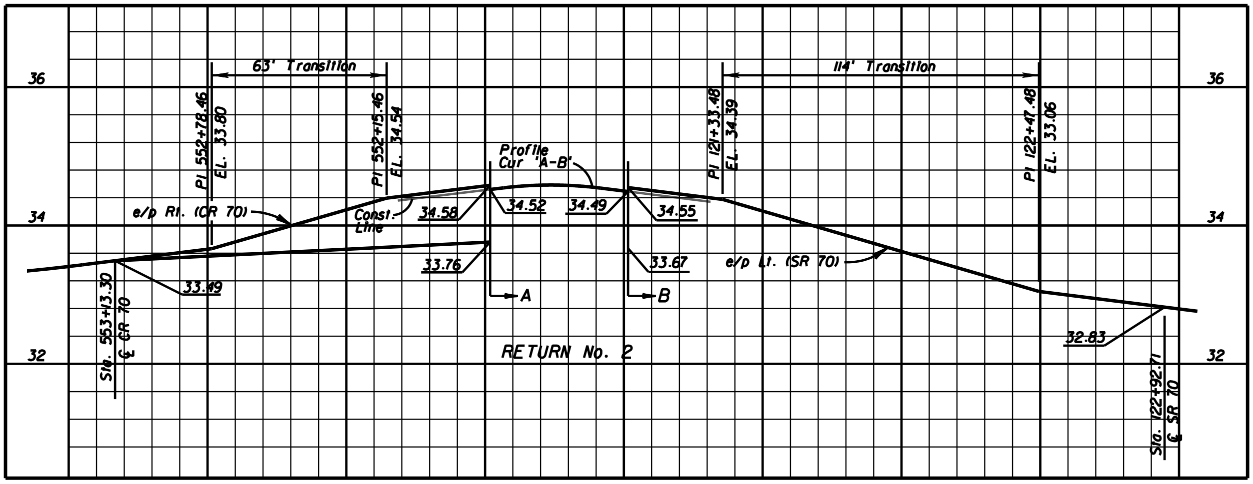
(34.52’ - 33.76’) / 20.45’ = 0.037’/ft.

This grade will provide some superelevation, meet the maximum slope for ADA requirements and meet the minimum gutter grade. Draw the return profile from the end of the return on CR 70 to section ‘A-A’ and label the elevation at section ‘A-A’. Calculate the elevation of the return at section ‘B-B’ by using a 0.037’/ft cross slope:

34.49’ - (21.94’ x 0.037) = 33.67’

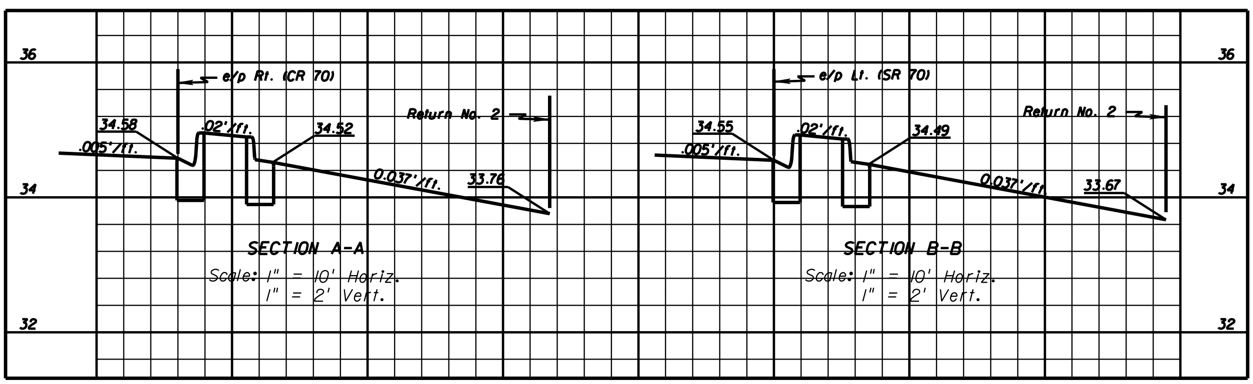
Label the elevation of the return at section 'B-B' as shown in ***Figure A-10***.

Figure A-10



Once the elevations at sections 'A-A' and 'B-B' are set, the cross sections can be completed by drawing in the cross slope of the pavement. The completed drawing is shown in ***Figure A-11***.

Figure A-11



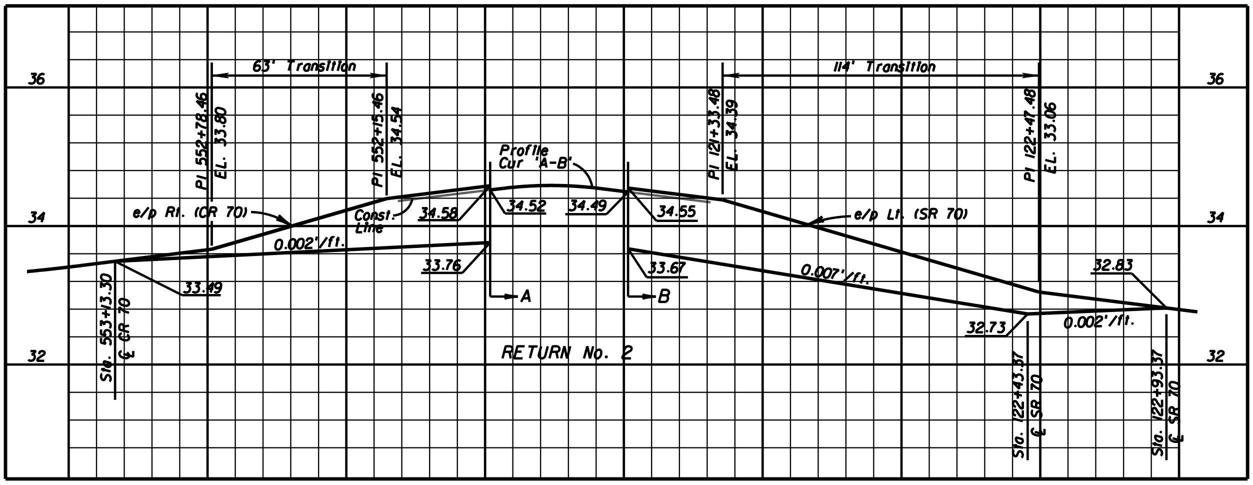
In order to avoid a flat cross slope in the vicinity of the right turn taper, it will be necessary to drain the water in the taper area back toward the intersection by setting a positive grade along the taper and installing a curb inlet at Sta. 122+43.37 Lt. Another solution would be to lengthen the taper to develop a gutter grade. However, this may not be practical with a grade as flat as 0.5% as the taper would be excessively long. This condition should always be checked so that water does not puddle in the vicinity of the taper.

Using a positive 0.2% grade from the beginning of the right turn taper at Sta. 122+43.37 to Sta. 122+93.37 plot and label the return profile elevations at these stations. Next, plot the return profile from section 'B-B' to the beginning of the taper at sta. 122+43.37 and check the profile grade. The grade is calculated as:

(32.73' - 33.67') / 143.95' x 100 = -0.65%

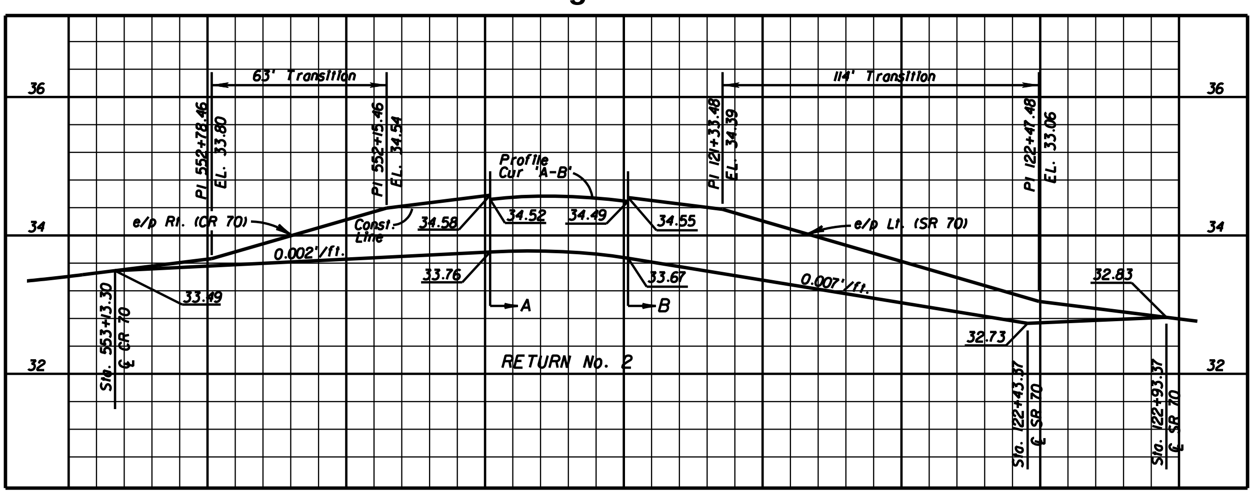
Therefore, the grade exceeds the minimum. The profile is shown in ***Figure A-12***.

Figure A-12



Finally, the return profile can be completed by constructing a vertical curve between sections 'A-A' and 'B-B' connecting the two grades as shown in ***Figure A-13***.

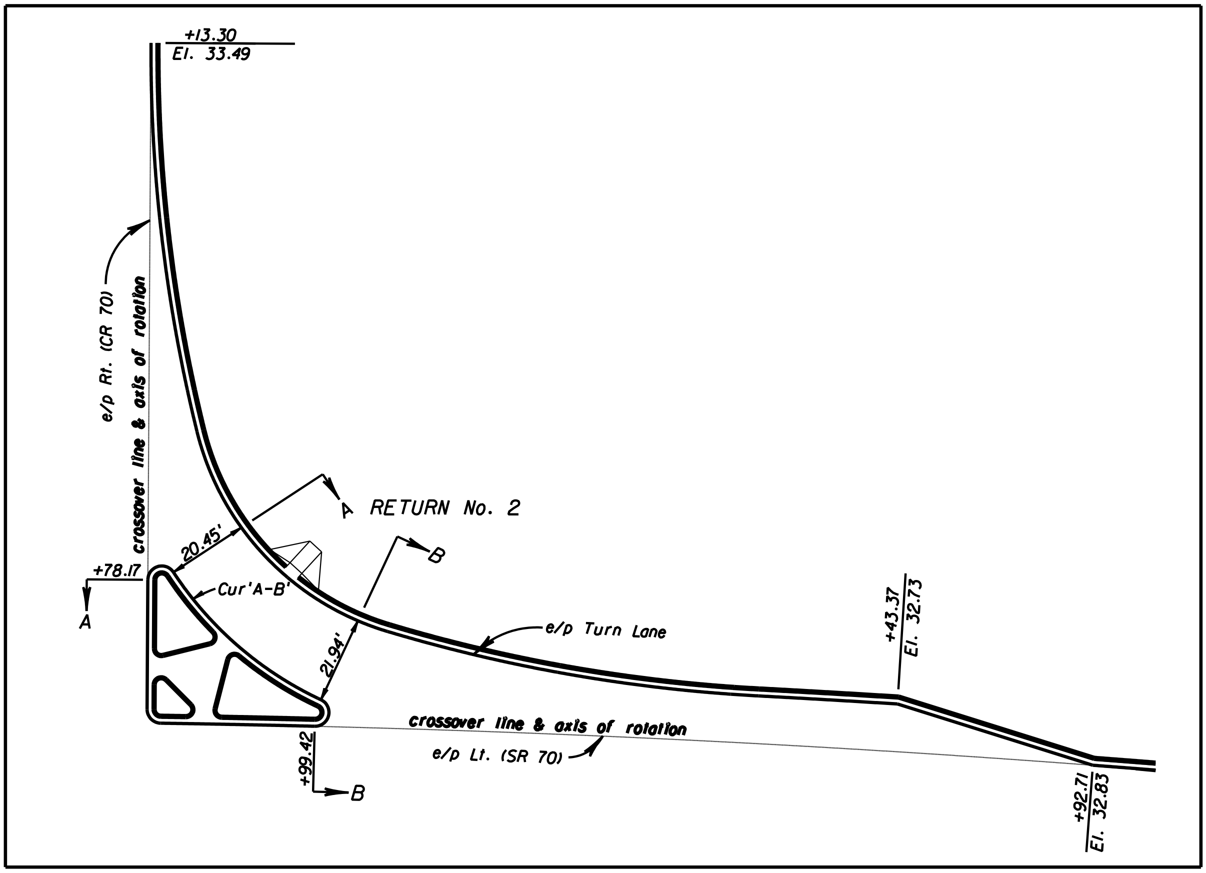
Figure A-13



<u>STEP 7: Check the Right Turn Lane Relative Gradient and Cross Slope Break-Over</u>

According to 2004 AASHTO Manual, page 176, "For appearance and comfort, the length of superelevation runoff should be based on a maximum acceptable difference between the longitudinal grades of the axis of rotation and the edge of pavement" as shown in ***Figure A-14***.

Figure A-14



A check should be made of the relative gradient for the turn lane. From the 2004 AASHTO Manual, page 643, table 9-44; the effective maximum relative gradient for a 12' wide auxiliary lane with a design speed of 45 mph is 0.54%. Referring to ***Figure A-14***, the relative gradient, Δ, of the right turn entrance can be calculated as:

Δ = (longitudinal grade of the axis of rotation) – (longitudinal grade of the edge of pavement)

Δ = ((34.39'-33.06')/114') x 100 – ((33.67'-32.73')/143.95') x 100 = 0.52% < 0.54% OK

Therefore, the relative gradient is acceptable and is in fact conservative since AASHTO allows for a greater relative gradient for lower design speeds and wider lane width. This example assumes a 45 mph design speed for the turn lane when in fact it will probably operate at lower speeds. In addition, the lane widens to 20' so the effective width is greater than 12'.

According to AASHTO (2004 – A Policy on Geometric Design of Highways and Streets, pg. 642) "there is a practical limit to the difference between the cross slope on the traveled way and that on the intersection curve. Too great a difference in cross slope may cause vehicles traveling over the cross-over crown line to sway sideways. When vehicles, particularly high-bodied trucks, cross the crown line at other than low speed and at an angle of about 10 to 40 degrees, the body throw may make vehicle control difficult." The maximum algebraic difference in cross slope at the crossover line is 5.0% in this case. The maximum cross slope difference occurs when the outside through lane has fully transitioned to a 0.50% cross slope and the right turn lane has transitioned to a 3.70% cross slope. The cross slope difference is calculated as:

3.70% - 0.50% = 3.20% < 5.00%

Therefore, the cross slope difference is acceptable along the entire cross-over crown line.

Again, referring to ***Figure A-14***, the relative gradient of the right turn exit can be calculated as:

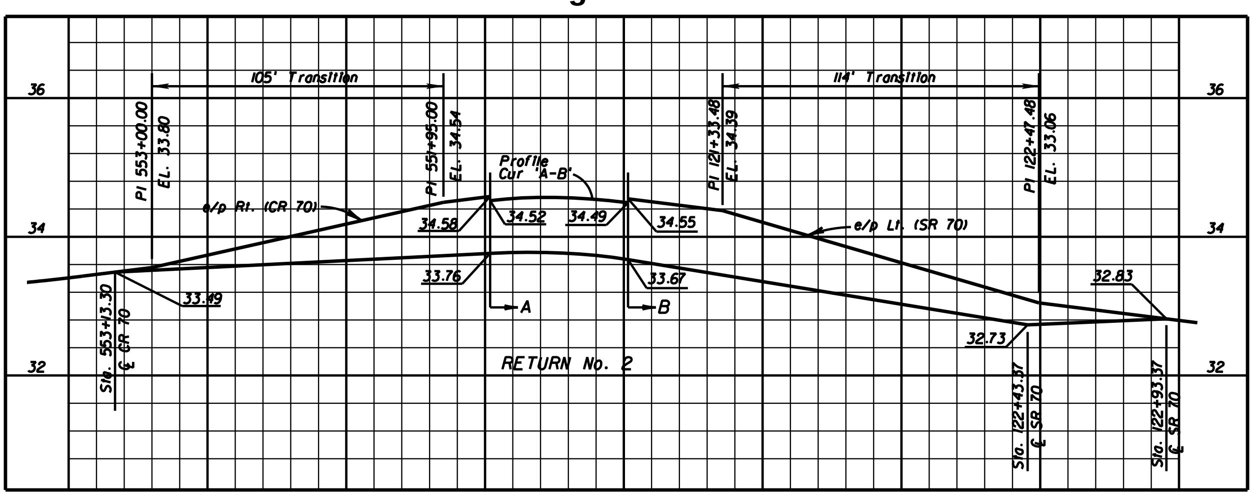
Δ = ((34.54 - 33.80)/63) x 100 - ((33.76 - 33.49)/135.13) x 100 = 0.97% > 0.54% NG

Therefore, the relative gradient must be reduced. This can be accomplished by either lengthening the transition or increasing the gutter grade, or both. It is preferable to lengthen the transition rather than increasing the gutter grade in order to maintain as much superelevation around the return as possible. Therefore, calculate the minimum transition length:

$TL_{min.}$ = ((34.54-33.80) x 100) / [((33.76-33.49)/135.3) x 100] + 0.54 = 100.06'.

We will use a 105' taper as shown in ***Figure A-15***.

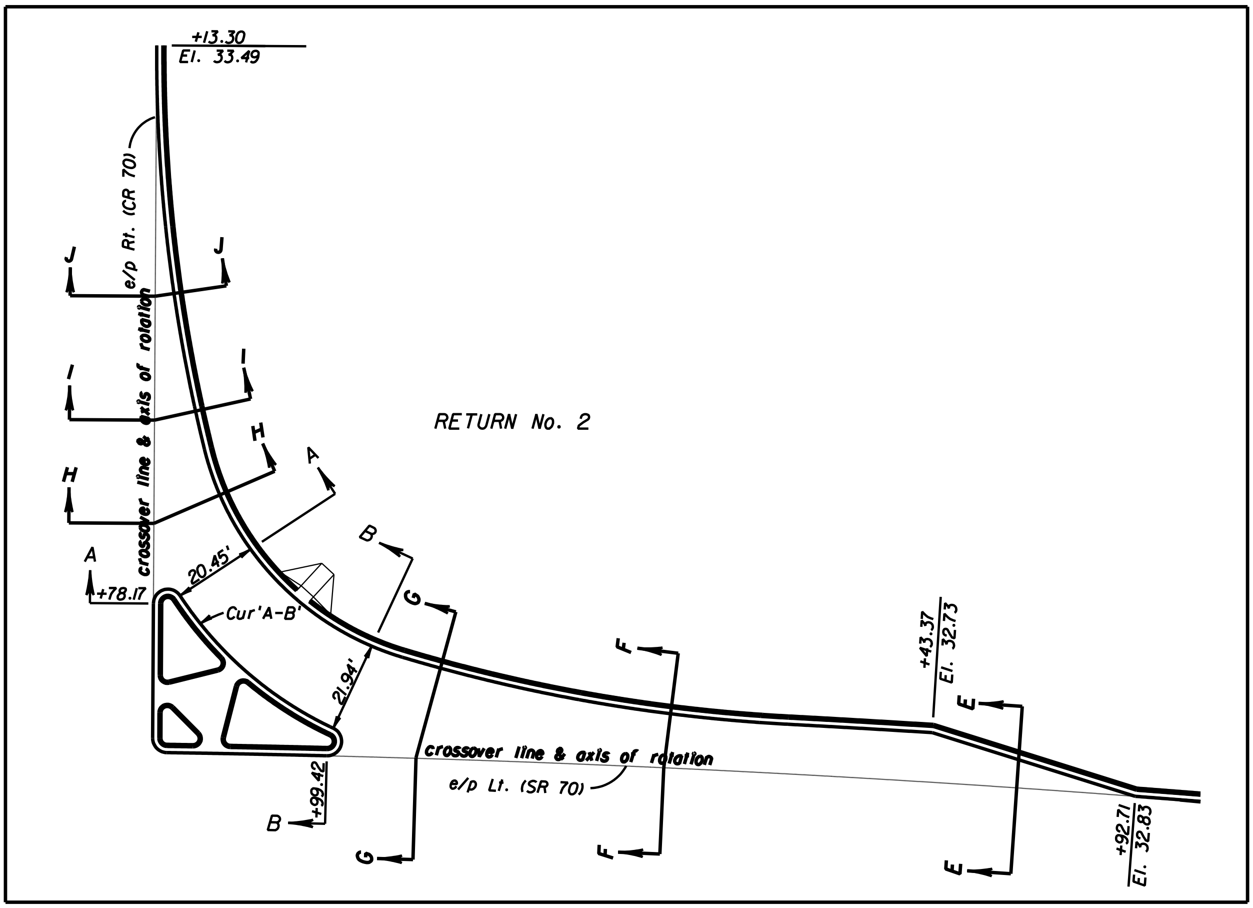
Figure A-15



The maximum cross slope difference for the right turn exit terminal again occurs where the outside through lane has fully transitioned to a 0.50% cross slope and the right turn lane has transitioned to a 3.70% cross slope. As with the entrance terminal the cross slope difference is acceptable along the entire cross-over crown line.

Final adjustments to the return profile will be made as a result of adjusting the cross slope of the turn lane to gradually superelevate from the entrance taper to the approach nose of the raised island, then gradually back to match the roadway cross slope at the end of the exit terminal. This exercise is accomplished by drawing sections at strategic "control points" along the superelevation transition, then fixing elevations on the return profile at these sections. A spline or 'best-fit' curve is then placed through the fixed points. This effort is made slightly more complicated by the fact that the edge of pavement of the through lane is in vertical transition and the turn lane is varying in width. Because of constraints within the intersection mathematically derived vertical curves are not always practical; however, according to AASHTO (2004 – A Policy on Geometric Design of Highways and Streets, pg. 649) "The final profile may not always produce the selected cross slope at all of the control points, but this problem is not serious as long as the cross-slope change is progressive and within the design control limits." The sections to be considered are shown in ***Figure A-16***.

Figure A-16



The sections at 'E-E' and 'J-J' should match the cross slope of the adjacent travel lane. The sections at 'G-G' and 'H-H' should have attained more than half of the superelevation rate with full superelevation reached at or just beyond the noses of the island. In this case we will develop approximately 80% of the desired superelevation at sections ‘G-G’ and ‘H-H’. At sections ‘F-F’ and ‘I-I’ the cross slope of the auxiliary lane should be steeper than the cross slope on the adjacent through traffic lane. These sections are shown in ***Figure A-17*** and ***Figure A-18***.

Figure A-17

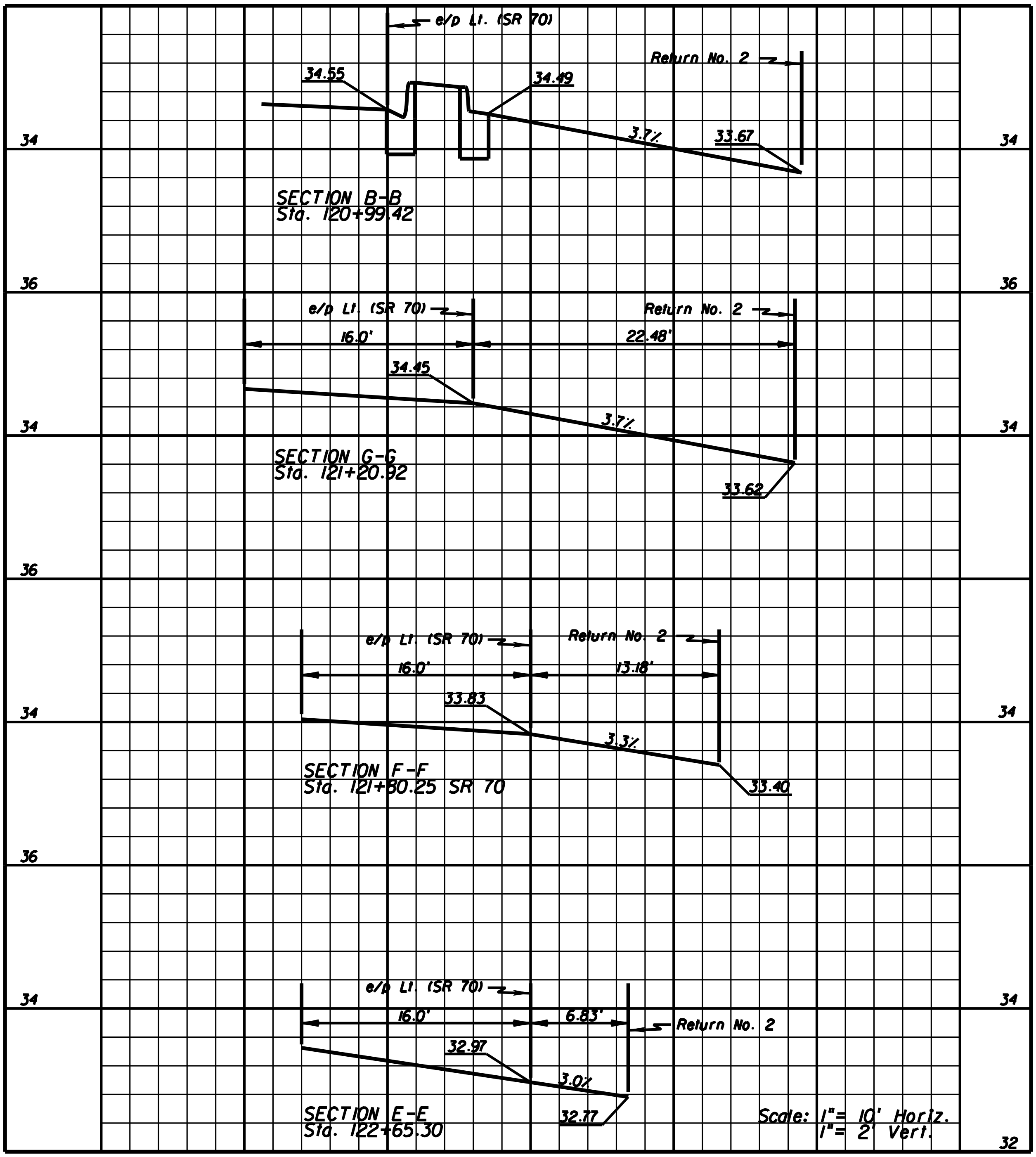
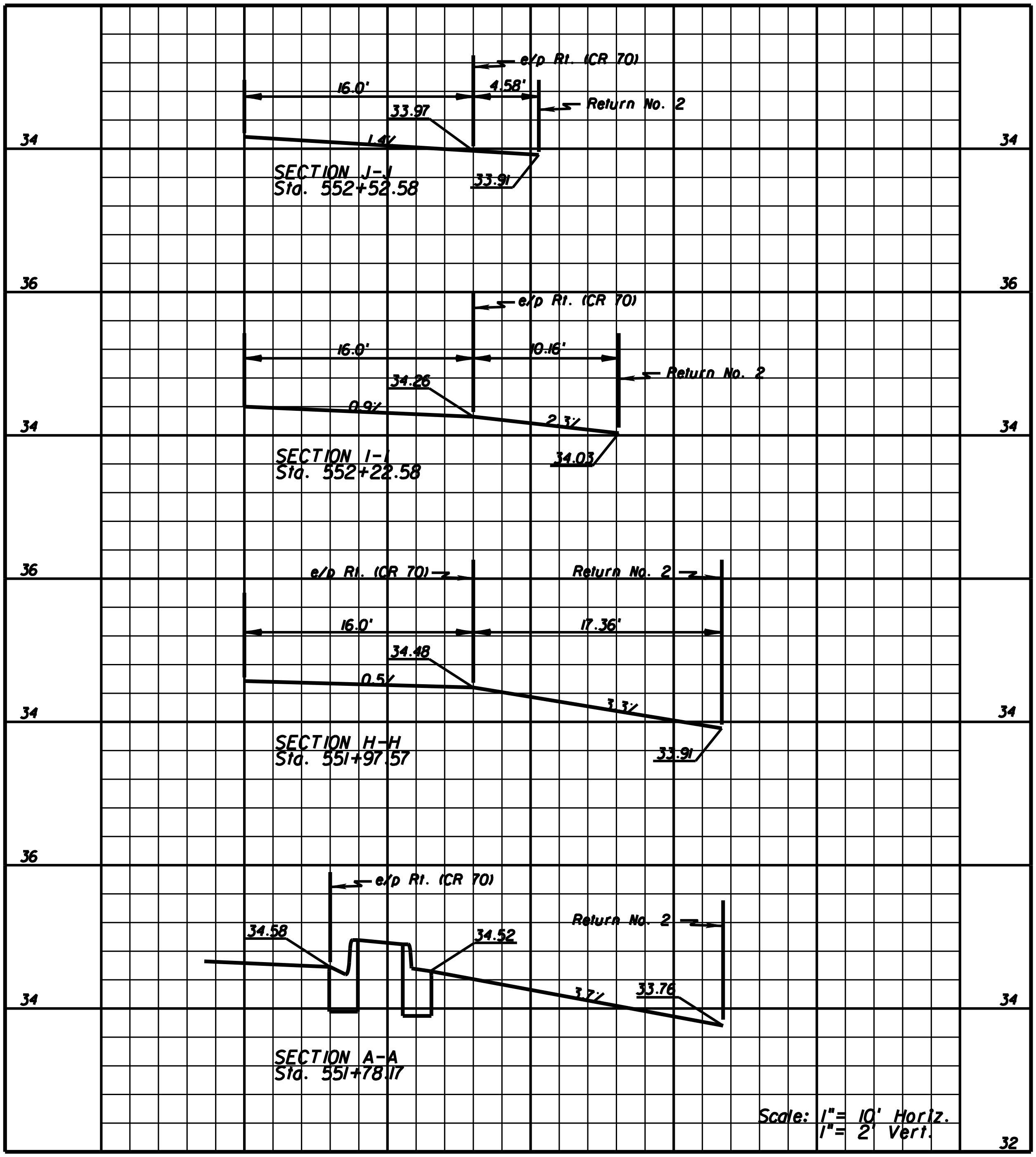
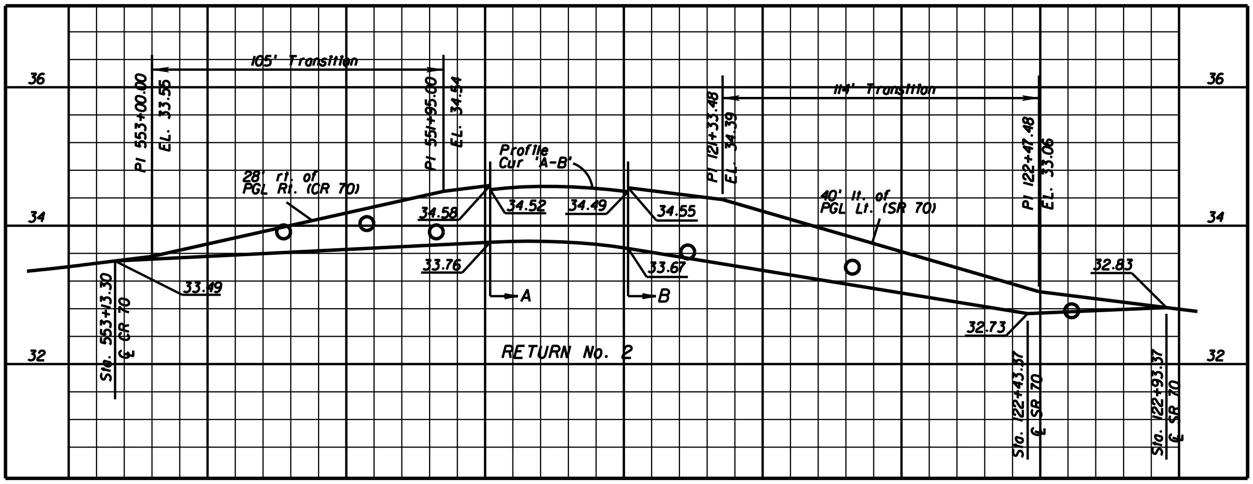


Figure A-18



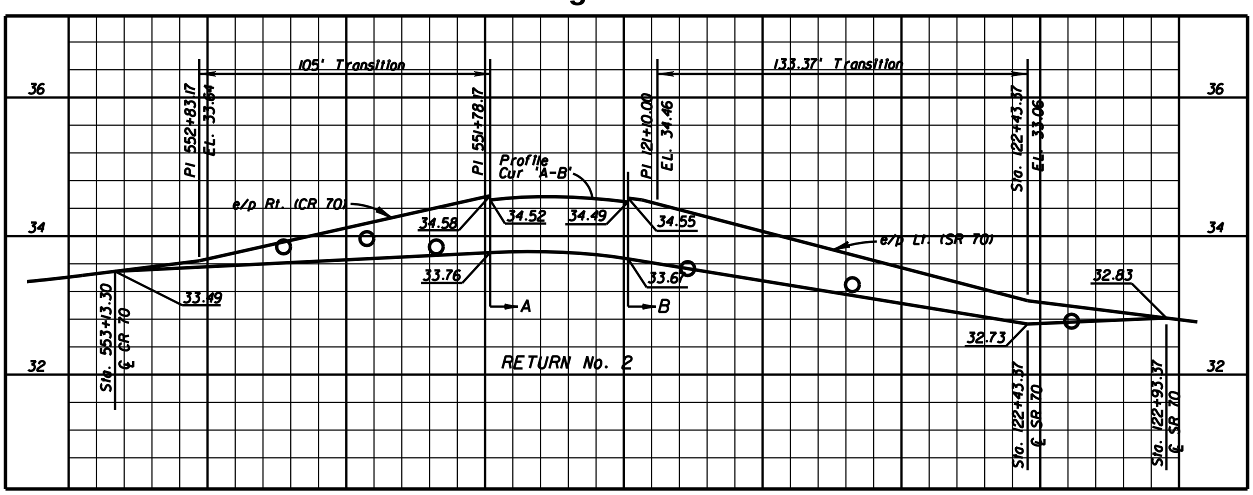
Next, plot the elevation of Return No. 2 at each section on the return profile drawing as shown in ***Figure A-19***.

Figure A-19



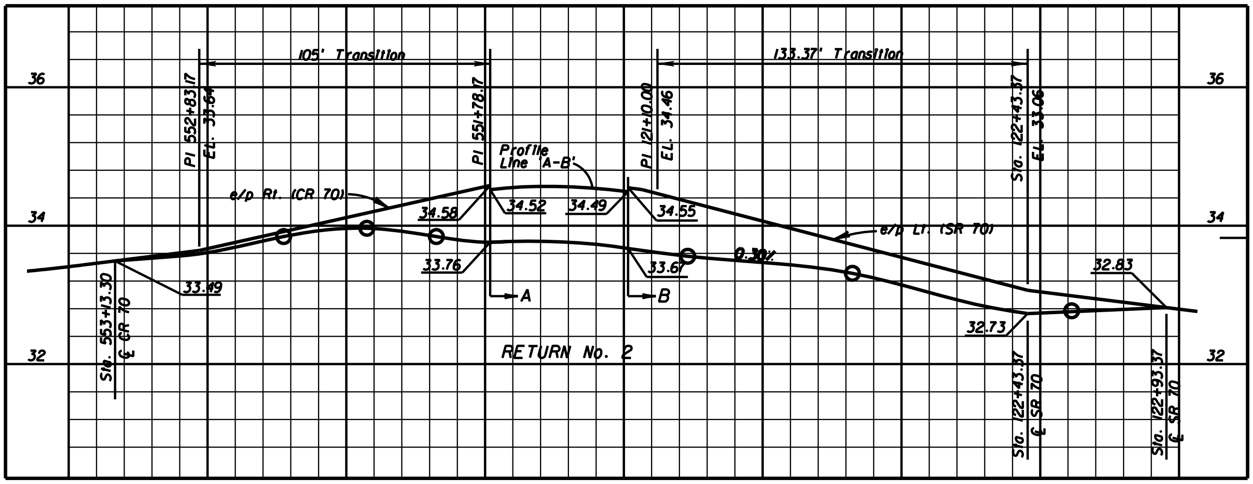
While maintaining the cross slope at each section, adjust the transitions toward the intersection in order to better match the preferred curb return profile. The entrance transition is lengthened and adjusted so that it begins at Sta. 121+10.00 and ends at Sta. 122+43.37. The return profile for the exit is going to deviate significantly from the preferred profile and also has a high point in the return that will require an inlet. The exit transition is adjusted forward and begins at Sta. 551+78.17 where section 'A-A' is located. The adjusted transitions are shown in ***Figure A-20***.

Figure A-20

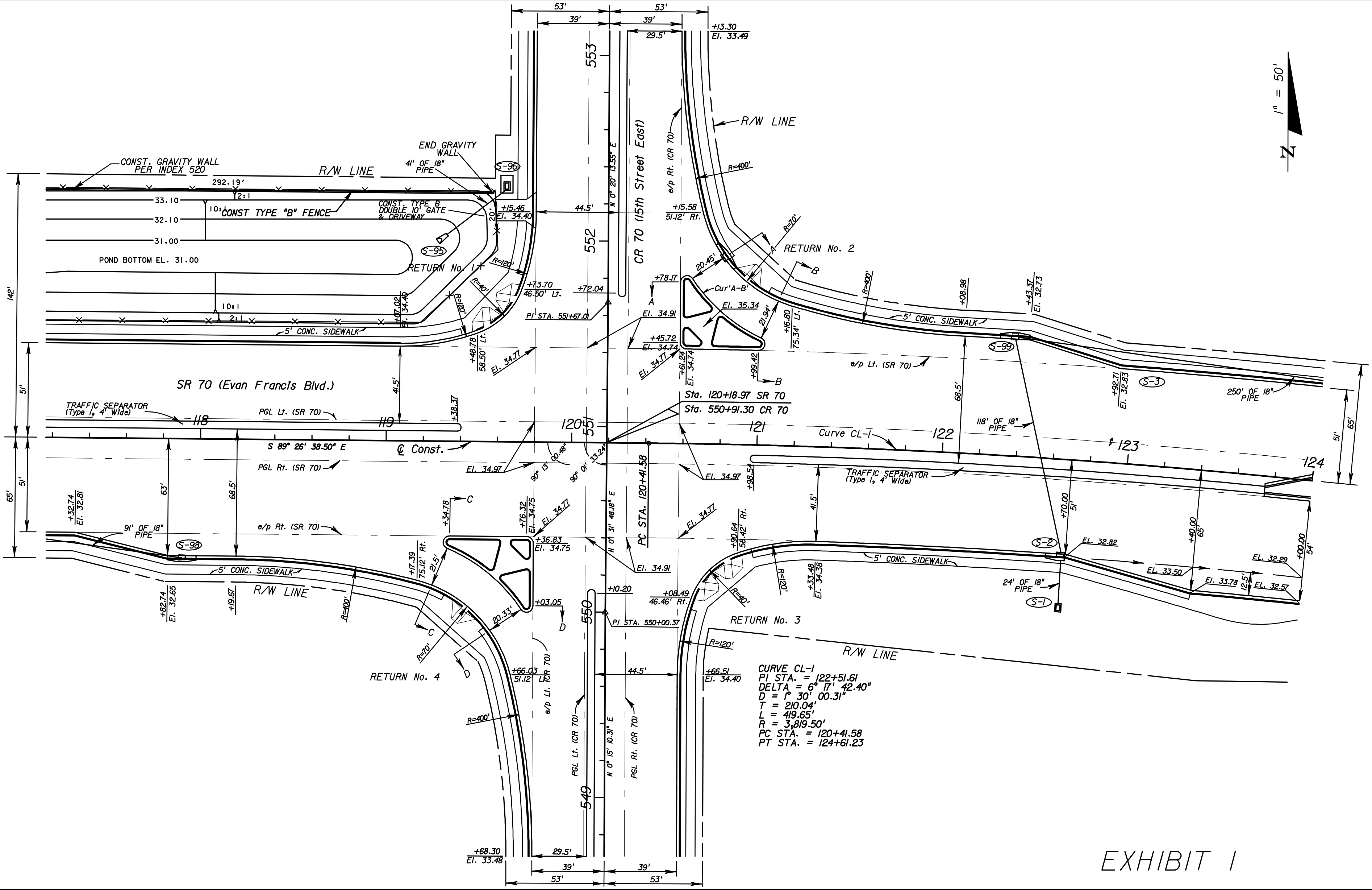


Draw a 'Best Fit' spline curve through the data points as shown in ***Figure A-21*** and check the final gutter grades.

Figure A-21



The minimum gutter grade on the return is 0.3% and will meets standards. An inlet will be placed in the return at Sta. 551+78.17. It will not interfere with the curb cut ramp and is out of the wheel path of the design vehicle.



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DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
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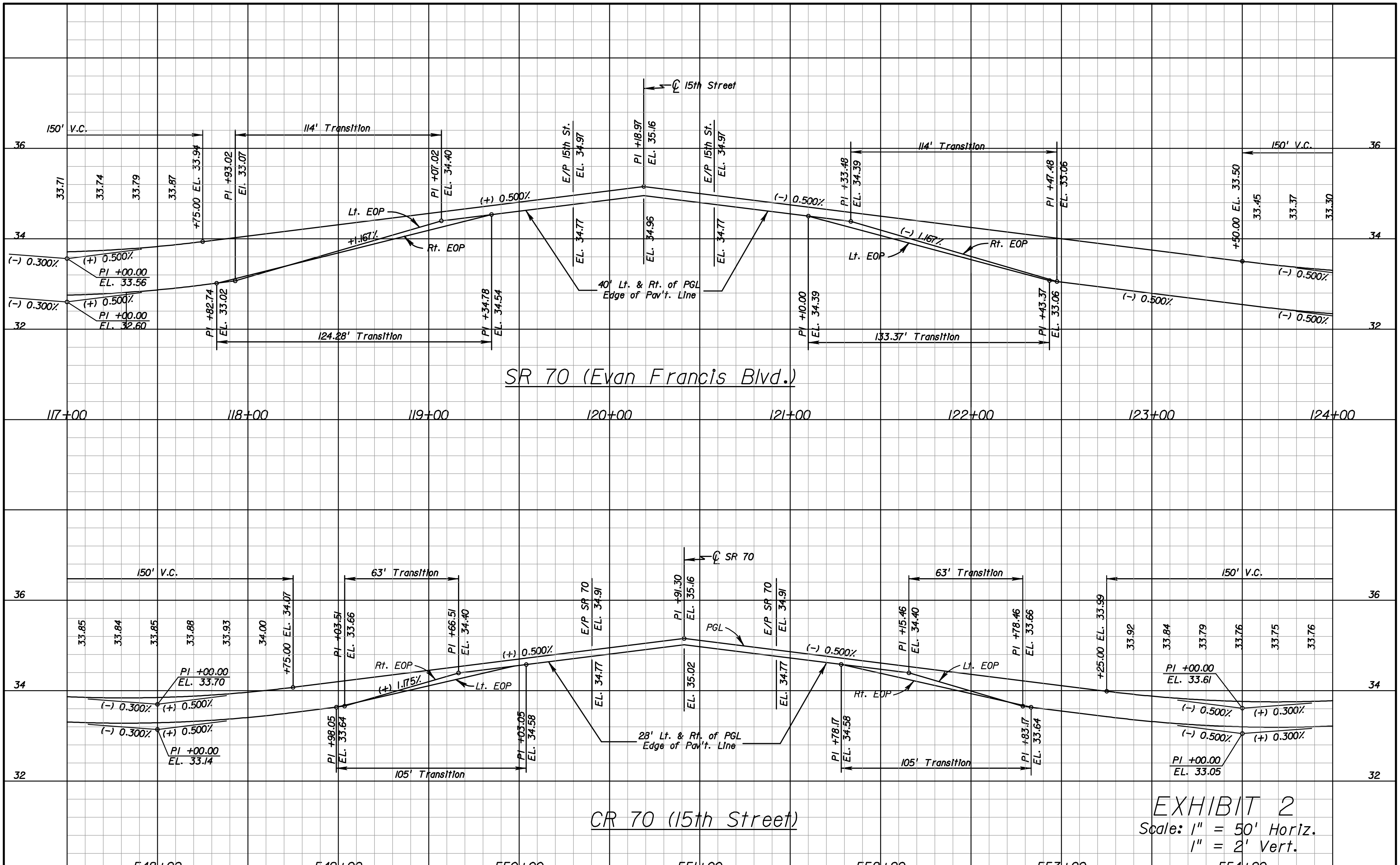
INTERSECTION DETAIL
SR 70 AT CR 70

SHEET NO.

$USER$ $DATE$ $TIME$ $FILE$



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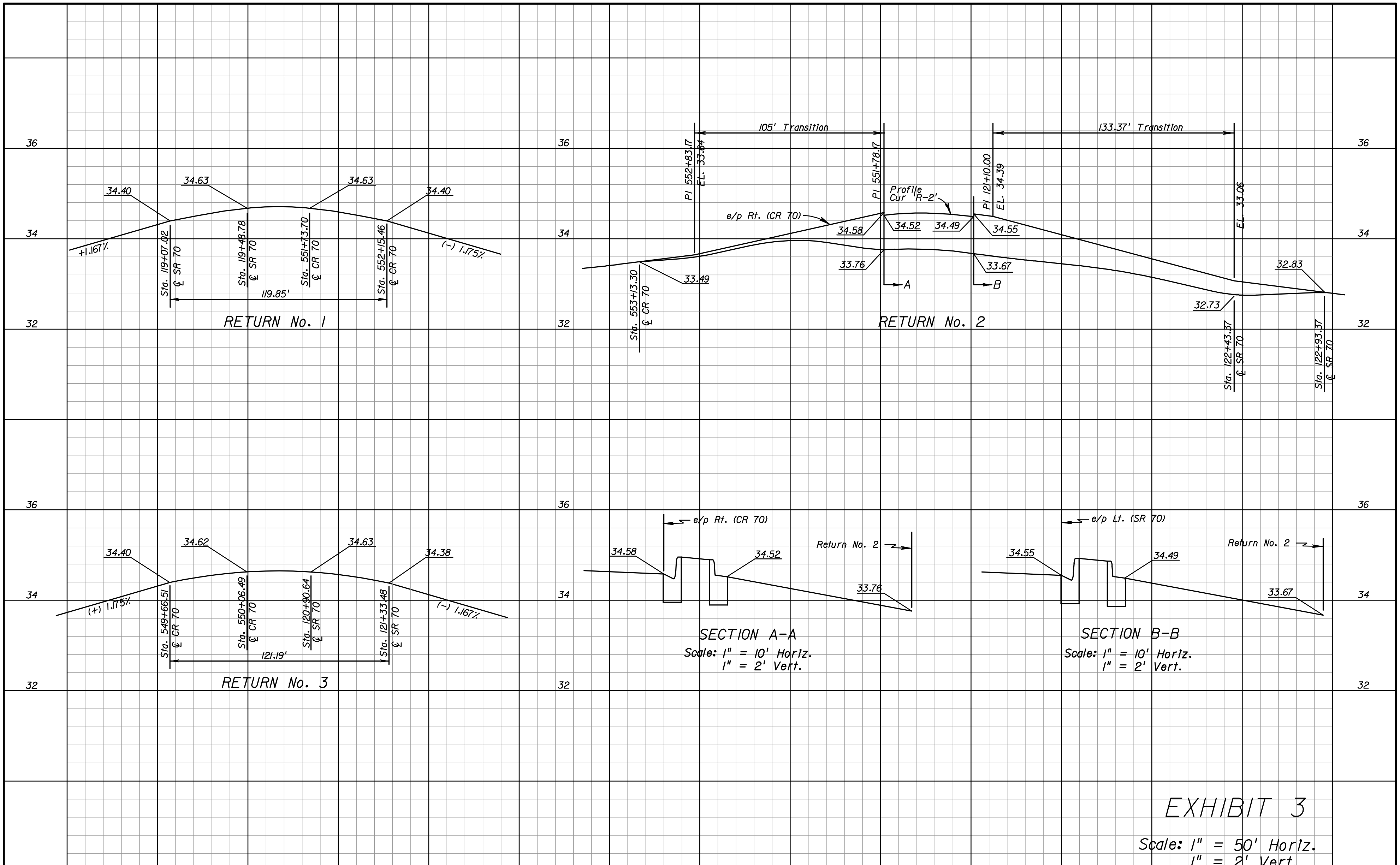

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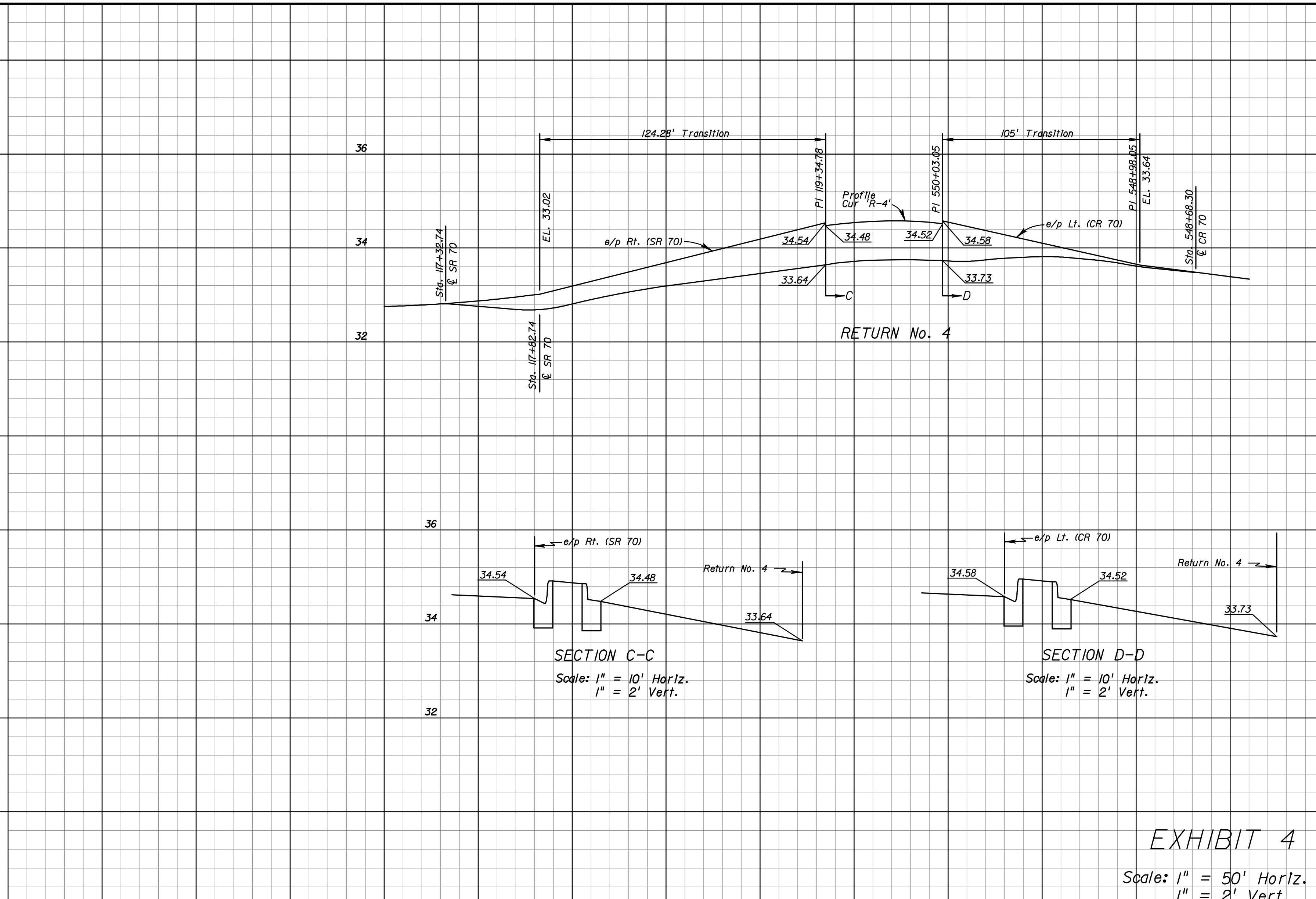

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