Florida Intersection Design Guide

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

Florida Department of Transportation
2007

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Chapter 1

Introduction

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Chapter 1

Introduction

1.0 General

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 Technical Advisory Committee 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:

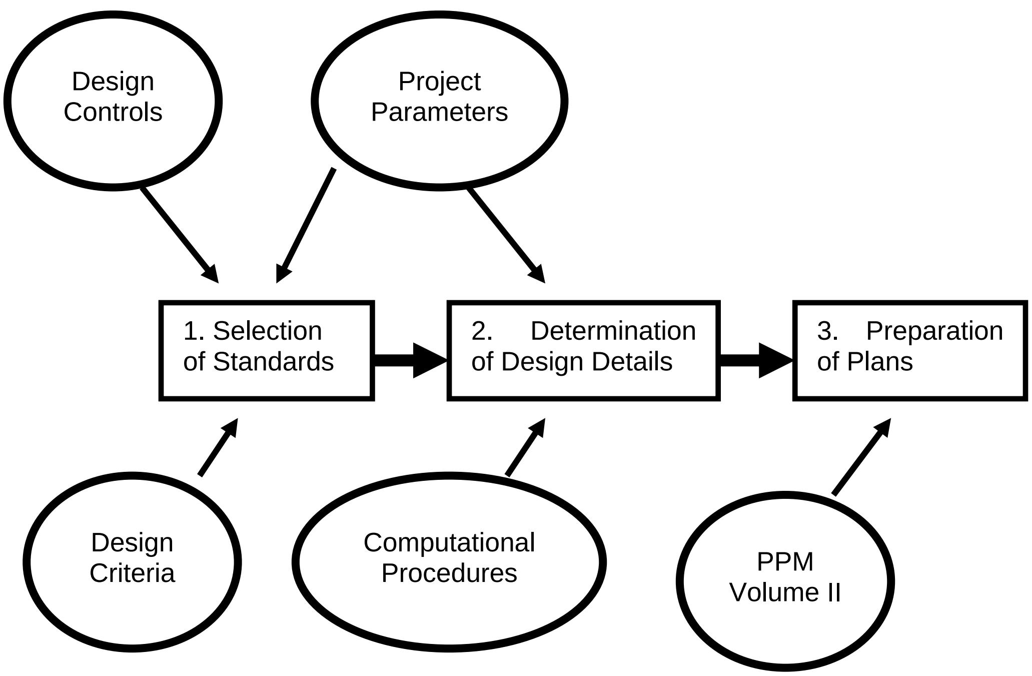
1. Safe and convenient operation for all road users, including cyclists and pedestrians;
2. Proper accessibility for pedestrians with special needs;
3. Adequate capacity for peak-hour demand on all movements;
4. Adequate maneuvering space for design vehicles;

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.

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 are also illustrated in ***Figure 1-1***. The process is described in the following sections.

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.

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.

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.

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.

> Example:
> The minimum pedestrian clearance time is based on an assumed walking speed of 4 ft/sec {MUTCD}.

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 publications described in the following sections 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: 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.

1.3.2 References Containing Design Guidelines

Florida Roundabout Guide: describes a process for justifying the construction of a roundabout and sets forth the principles and criteria that govern the design.

Trail Intersection Design Handbook: 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***: 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***: 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: 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 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.

1.3.4 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.

1.4 Intersection Design Issues

The term “issue” is used in many different contexts. Webster’s dictionary offers nine separate definitions of this word. The definition of interest here 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 the Technical Advisory Committee. 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

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Chapter 2

Intersection Design Concepts

2.0 General

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.

The functional area on the approach to an intersection or driveway consists of three basic elements: (1) perception reaction distance, (2) maneuver distance and (3) queue-storage distance. 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.

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***.

Table 2-1 Summary of Intersection Characteristics

Physical characteristics	Traveled roadway Curbs Sidewalks Medians Islands 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
Site characteristics	Roadway classification Site location Roadside development Institutional proximity (schools, etc.)
Road User Characteristics	Age Special requirements

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.

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. 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*** consider a bicycle to be a vehicle. In the traveled roadway, a bicycle is always considered as a vehicle. In crosswalks, the ***Florida Statutes*** allow the cyclist the option of assuming the rights and duties of a pedestrian. The accommodations for bicycles at intersections must recognize this option.
3. Bicycle-pedestrian conflicts: Because they assume the identity of a 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 to the first arriving vehicle at an uncontrolled intersection or to the vehicle on the right in cases of simultaneous arrival. ***Florida Statutes, Section 316.123*** states the same rule for all-way stop control (AWSC).
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 vehicles in conflict with left turns and ***Florida Statutes, Section 316.123*** governs vehicles on approaches controlled by stop or yield signs.
3. Alternating Priority (signals): The assignment of right-of-way by traffic signals is covered in ***Florida Statutes, Section 316.075***.
4. 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 large traffic circles and some types of freeway interchanges.
5. Grade Separation: The conflict between movements is automatically resolved 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. Two-way stop control (TWSC): This mode requires minimal justification and there are no numerical warrants to be applied. The ***MUTCD*** suggests that TWSC control may be warranted when one route is clearly more important than the other and when conditions indicate the need.
3. All-way stop control (AWSC): The justification of this mode is subject to numerical traffic volume warrants presented in the ***MUTCD***. Additional justification categories include locations with high crash rates and locations where traffic signals are warranted but have not yet been installed.
4. Signal Control: The ***MUTCD*** enumerates eight 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. The ***MUTCD*** discourages the installation of a signal if it would seriously disrupt progressive traffic flow.

> STOP signs shall not be used in conjunction with any traffic control signal except:
> 1. If the signal indication for an approach is a 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 ***{MUTCD}***.

> **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
>
> "The satisfaction of a traffic signal warrant or warrants shall not in itself require the installation of a traffic control signal."

5. Roundabout: Roundabouts are gaining popularity in Florida because they have been demonstrated to accommodate moderate traffic volumes with lower delays and improved safety compared to a traffic signal. The Florida Roundabout Guide describes a procedure for justification of a roundabout. It also sets forth the criteria to be used in determining design standards for roundabouts on the SHS. The roundabout justification procedure is duplicated in the FDOT ***MUTS***.

> Consideration should be given to providing alternatives to traffic control signals even if one or more of the signal warrants has been satisfied. ***{MUTCD}***

2.2.4 Intersection Control Mode Selection Procedures

The FDOT ***MUTS*** prescribes justification studies for signals and roundabouts. 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.

> An engineering study of traffic conditions, pedestrian characteristics and physical characteristics of the location shall be performed to determine whether installation of a traffic control signal is justified at a particular location. The investigation of the need for a traffic control signal shall include an analysis of the applicable factors contained in the traffic signal warrants and other factors related to existing operation and safety at the study location. ***{MUTCD}***

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 flashing beacons to supplement STOP sign control or provide advance warning;
5. Adding one or more lanes on a minor-street approach to reduce the number of vehicles per lane on the approach;
6. Revising the geometrics at the intersection to channelize vehicular movements and reduce the time required for a vehicle to complete a movement;
7. Installing roadway lighting if a disproportionate number of crashes occur at night;
8. Restricting one or more turning movements, perhaps on a time-of-day basis, if alternate routes are available;
9. Installing multi-way STOP sign control if the warrant is satisfied;

> For purposes of signal warrant evaluation, bicyclists may be counted as either vehicles or pedestrians. ***{MUTCD}***

10. Installing a roundabout and
11. Employing other alternatives, depending on conditions, at the intersection.

2.3 Estimation of Capacity

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 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 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 FDOT as the preferred capacity analysis technique. Software is available that provides a faithful implementation of the ***HCM*** procedures.

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, as defined in Chapter 2 (sec/veh), 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.

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 Signalized Intersections

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).

The LOS at a signal is defined in the ***HCM, Chapter 10, Urban Street Concepts*** in terms of the average control delay experienced by each vehicle on the approach. Threshold values are provided to distinguish among levels of service.

2.3.1.2 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 10 of the HCM*** as a gap acceptance process. The gap acceptance parameters include:

1. The critical gap, which is defined as the minimum gap in the major street flow that the average driver is willing to accept and
2. The follow-up time, which is defined as the incremental gap time required by subsequent vehicles entering the same gap.

These two parameters determine the number of vehicles that may enter a gap of a given length. The computation of the approach capacity requires some knowledge or assumptions about the distribution of gaps in the major flow. The ***HCM*** sets forth the assumptions and computational procedures for estimating capacity and level of service based on this modeling concept for two-way stop control. Separate procedures are described for all-way stop control.

2.3.1.3 Roundabouts

Roundabouts represent a newer traffic control mode that is finding increasing acceptance in Florida. Roundabout performance analysis is covered in the ***Florida and Federal Highway Administration (FHWA) Roundabout Guides***. The HCM provides a method for approximating the capacity of a roundabout as a series of interconnected yield-controlled junctions. A more detailed roundabout gap acceptance model, developed in Australia, uses an empirical procedure to determine the gap acceptance parameters (critical gap and follow-up time) as a function of the roundabout geometrics.

The Australian procedure has been implemented, with some modifications, in a software product called ***aaSIDRA***.

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:

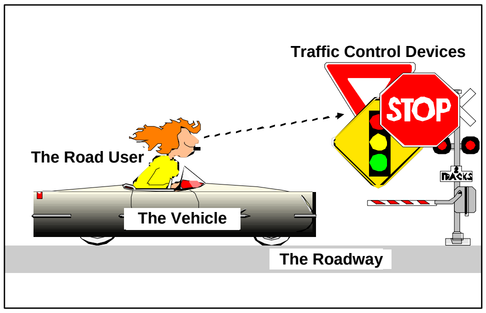
1. Incident delay: The additional delay caused by the occurrence of an incident that reduces the capacity of the facility.
2. Control delay: The delay imposed by the traffic control. This component may be further characterized by the mode of operation, i.e., signal delay, stop sign delay or roundabout 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 2-1.***

Figure 2-1 Intersection Operation



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 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:

<u>The Design Vehicle</u>: 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.

<u>Acceleration Performance</u>: 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.

<u>Braking Performance</u>: 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.

2.5.2 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.

Guidance Task: Highway design and traffic operations have the greatest effect on guidance. Road user performance can be improved by paying proper attention to lane placement and road following, car following, overtaking and passing and other guidance activities, (i.e., merging, lane changing, avoidance of pedestrians and response to traffic control devices, etc.).

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.

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***.

2.5.3 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:

316.081 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;

316.151 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

316.0765 Lane direction control signals.

The current version of these sections is 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. Stormwater 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:

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***.

2. Signal Controller Operation

In theory, a signal controller may be either pretimed or traffic-actuated. Pretimed 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. Pretimed 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 been satisfied, 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.

3. 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.

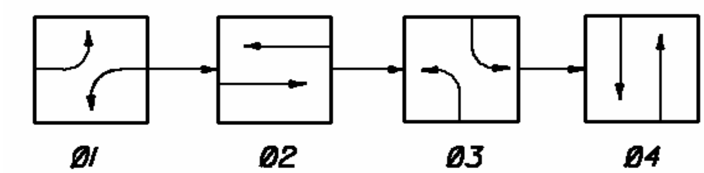
Vehicular presence during the red interval will place a demand for a green signal to be displayed. The detector is ignored during the display of the minimum green interval, but after this interval has expired, the presence of a vehicle will cause the phase to be extended until a gap of the prescribed 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 2-2*** and ***Figure 2-3***. 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 observe vehicular or pedestrian presence.

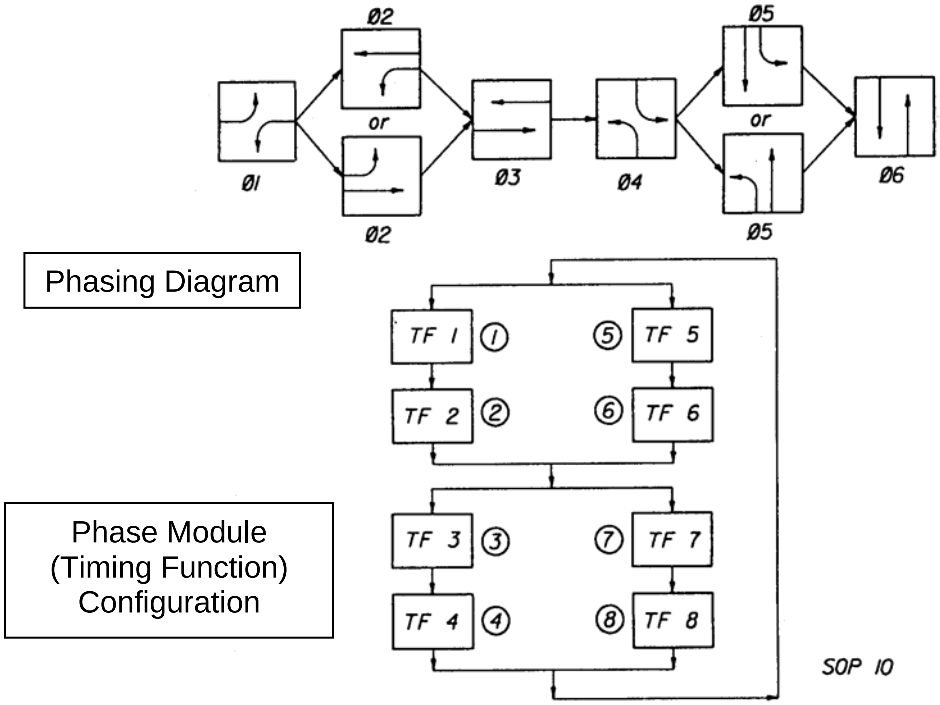
This simple sequence is known as single-ring sequential operation. In its most elementary form, only two phase modules will be involved in the sequence, with each controlling an opposing pair of approaches. Additional modules may be added to a single ring operation to provide exclusive phases for left turns or other minor movements. An example of single ring operation is presented in ***Figure 2-2***, with four phases displayed sequentially as the terminology suggests. Note that each phase module controls two approaches that move simultaneously.

Figure 2-2 Phasing Diagram for Single-Ring Sequential Signal Phasing



Most traffic-actuated intersections in Florida use a more complex scheme known as dual-ring concurrent operation, which is depicted in ***Figure 2-3***. Note that Phases 2 and 5 in the phasing diagram offer two alternatives, only one of which may be displayed on any cycle.

Figure 2-3 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. 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.

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 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.

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. Adjacent land uses, especially if the design is proposed as 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 and a suburban or rural intersection are illustrated in ***Figure 2-4*** 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 ***Chapter 3***. The traffic operational aspects will be covered in ***Chapters 4 and 5***. Note that the design details shown in this figure are intended for purposes of general illustration and should not be interpreted as specific recommendations.

2.10 Determining the Basic Intersection Layout

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.

> Adequate roadway capacity should be provided at a signalized location. ***{MUTCD}***

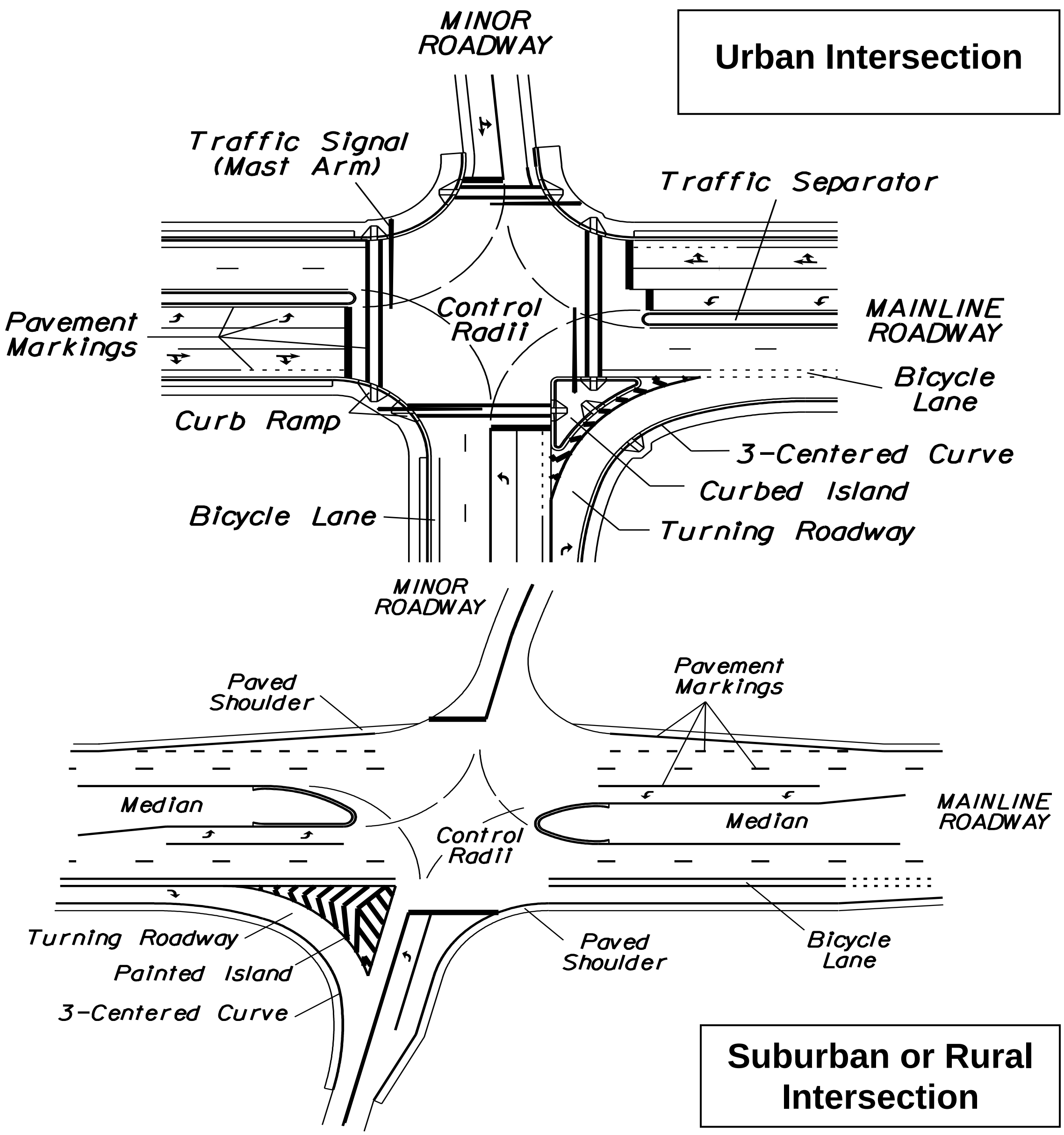
The basic configuration begins with the number of through lanes available on the two intersecting streets. Additional lanes will often be necessary to accommodate left and right turns. In some cases a widening of the intersection may be required to accommodate additional through lanes on the approach, with a lane drop on

> 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}***

the exit. Multiple left or right turn lanes may be required in addition to channelization for left or right turns.

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 will usually be the principal determinant, the tradeoff between capacity for vehicles and safety for pedestrians and bicyclists cannot be ignored. While this tradeoff is difficult to quantify, the negative effect of added turning lanes on pedestrians and bicyclists is widely recognized. In general, any provision that requires widening of the intersection should be justified on the basis of its demonstrated importance to the capacity of the intersection and mitigation of its adverse effects should be considered.

Figure 2-4 Typical Layout for an Urban and Rural Intersection



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Chapter 3

Geometric Design

Figures

Tables

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Chapter 3

Geometric Design

3.0 General

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. These elements provide the framework for the design of other highway elements including traffic control devices, roadway lighting, pavement design, drainage and structural design.

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. Intersections should be as close to right angle as practical.
6. Sight distance should be sufficient for crossing and turning maneuvers.
7. The intersection layout should encourage smooth flow and discourage wrong-way movements.
8. Auxiliary turn lanes should be provided 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. Special consideration should be given to requirements for accommodating bicycle and pedestrian movements.

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.

The use of circular intersections with all yield control in this country has been limited. However, a special subset of circular intersections that conform to a prescribed set of principles and standards, generally referred to as roundabouts, has been considered in recent years as a viable alternative to conventional intersections.

The geometric design criteria presented in this chapter apply only to non-circular at-grade intersections. The geometric design criteria for roundabouts can be found in ***Chapter 4 of the Florida Roundabout Guide***. The ***Florida Roundabout Guide*** also provides a series of justification categories for selecting a roundabout over the conventional intersections.

3.2.2 Functional Classification

Functional classification is the assignment of roads into systems according to the character of service they provide in relation to the total road network. The three main categories of roads are: arterials, collectors and locals. The ***PPM*** provides the following definition for each.

1. Arterial: Divided or undivided, relatively continuous routes that primarily serve through traffic, high traffic volumes and long average trip lengths. Traffic movement is of primary importance, with abutting land access of secondary importance. Arterials, classified as either urban or rural, include expressways without full control of access, US numbered routes and principal state routes.
2. Collector: Divided or undivided routes that serve to link arterial routes with local roads or major traffic generators. They serve as transition link between mobility needs and land use needs. Collectors include minor state routes, major county roads and major urban and suburban streets.
3. Local: Routes that provide high access to abutting property, low average traffic volumes, short average trip lengths and on which through traffic movements are not of primary importance. Local roads include minor county roads, minor urban and suburban subdivision streets and graded or unimproved roads.

The SHS roadways primarily consist of arterials and collectors, but intersection design on the SHS will involve all classifications.

AASHTO's ***A Policy on Geometric Design of Highways and Streets (AASHTO Green Book)*** contains a thorough discussion on roadway functional classifications. ***Florida Statutes, Title XXVI, Chapters 334, 335 and 336*** give similar definitions and establish classifications for road design in the State of Florida.

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]***.

3.2.3 Intersection Control

At-grade intersections on the SHS are typically controlled by stop signs (i.e., stop controlled) or traffic signals (i.e., signalized). The type of intersection control has a direct effect on a number of geometric design features, including sight distance and storage length of auxiliary lanes.

3.2.4 Area Type

Area type is typically classified as urban 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 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.

3.3 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. It must be selected very early in the design process. The selection of an appropriate design speed must consider many factors. The ***AASHTO Green Book*** has a thorough discussion on design speed and these factors.

3.3.1 Through Movements

The mainline design speed will influence the design elements of the intersection such as location and design of islands, taper lengths, type of control and sight distance requirements. ***Table 3-1*** provides a recommended range of design speeds for new construction and reconstruction projects on the SHS except for facilities on the Florida Intrastate Highway System (FIHS)/Strategic Intermodal System (SIS). Design Speed for facilities on the FIHS/SIS (including SIS Highway Corridors, Emerging SIS Highway Corridors, SIS Highway Intermodal Connectors and Emerging SIS Highway Intermodal Connectors) shall meet or exceed the values in ***Table 3-2***.

Table 3-1 Mainline Design Speed for the State Highway System (Non-FIHS/SIS Facilities)

Type Facility (Non-FIHS/SIS)		Design Speed (mph)
Arterials	Rural	55-70
	Urban	40-60
Collectors	Rural	55-65
	Urban	35-50
Intersecting Locals	Rural	30–50
	Urban	20-30
TDLC		30-40

Table 3-2 Minimum Mainline Design Speed for FIHS/SIS Facilities

Type Facility (FIHS/SIS)		Design Speed (mph)
Arterials	Rural	65
	Urban and Urbanized	50

3.3.2 Turning Movements

Vehicles turning at intersections designed for minimum-radius turns have to operate at low speeds of less than 10 mph. While it is 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, 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 design of an intersection is significantly affected by the type of design vehicle, including horizontal and vertical alignments, lane widths, turning radii, 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.

Modern computer software, such as AutoTURN, 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 output of AutoTURN for testing the turn radius adequacy for a WB-62FL design vehicle.

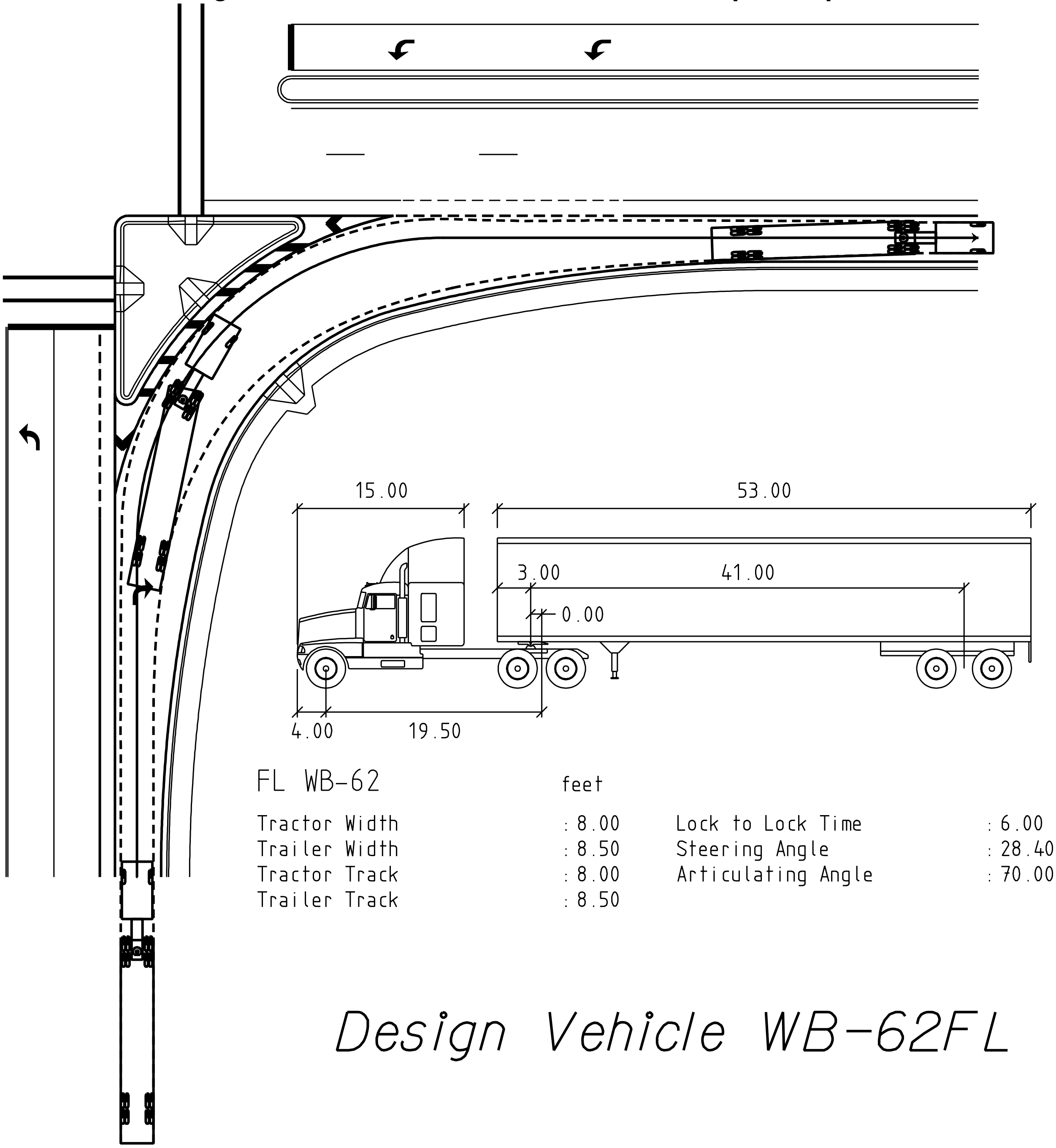
Table 3-3 Dimensions, in Feet, of Design Vehicles [AASHTO 2004]

Design Vehicle Type	Symbol	Overall			Overhang	
		Height	Width	Length	Front	Rear
Passenger Car	P	4.25	7	19	3	5
Single Unit Truck	SU	11-13.5	8	30	4	6
Intercity Bus	BUS-40	12	8.5	40	6	6.3
Intercity Bus	BUS-45	12	8.5	45	6	8.5
City Transit Bus	CITY-BUS	10.5	8.5	40	7	8
Conventional School Bus	S-BUS 36	10.5	8	35.8	2.5	12
Large School Bus	S-BUS 40	10.5	8	40	7	13
Articulated Bus	A-BUS	11	8.5	60	8.6	10
Intermediate Semitrailer	WB-40	13.5	8	45.5	3	2.5
Intermediate Semitrailer	WB-50	13.5	8.5	55	3	2
Interstate Semitrailer	WB-62*	13.5	8.5	68.5	4	2.5
Interstate Semitrailer	WB-65** or WB-67	13.5	8.5	73.5	4	4.5-2.5
Double Bottom Semitrailer/Trailer	WB-67D	13.5	8.5	73.3	2.33	3
Triple Semitrailer / Trailers	WB-100T	13.5	8.5	104.8	2.33	3
Turnpike Double Semitrailer/Trailer	WB-109D*	13.5	8.5	114	2.33	2.5
Motor Home	MH	12	8	30	4	6
Car and Camper Trailer	P/T	10	8	48.7	3	10
Car and Boat Trailer	P/B	----	8	42	3	8
Motor Home and Boat Trailer	MH/B	12	8	53	4	8

* = Design vehicle with 48ft. trailer as adopted in 1982 Surface Transportation Assistance Act (STAA)

** = Design vehicle with 53ft. trailer as grandfathered in with 1982 STAA

Figure 3-1 AutoTURN Turn Radius Example Output



3.5 Pedestrian Considerations

Provisions for pedestrian and bicycle traffic should be incorporated into the original intersection design. All new or major reconstruction projects should be designed with the consideration that pedestrians and bicyclists will use them. Decisions on appropriate pedestrian and bicycle facilities shall be determined with input from the District Pedestrian/Bicycle Coordinators and District Americans with Disabilities Act (ADA) Coordinators.

Return radii at an intersection must balance the needs of the pedestrian and the design vehicle. Large radii are needed to accommodate a vehicle's turning ability while small radii are needed to minimize the crossing distance for pedestrians. In urban areas, where a parking lane is present, curb extensions may be used to minimize the crossing distance.

Pedestrian facilities must be designed in accordance with ADA to accommodate those who 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 Considerations

When on-street bicycle lanes and/or off-street shared use paths enter an intersection, the design of the intersection should be modified accordingly. In addition, even in locations where there are no bicycle facilities, the inclusion of bicycle lanes on intersection improvement projects must be considered as these intersections may be excepted out of later roadway projects.

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 ***Figure 2-4*** provides one example of this treatment. Standard drawings for various bicycle lane configurations are provided in the ***Design Standards, Index 17346***.

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

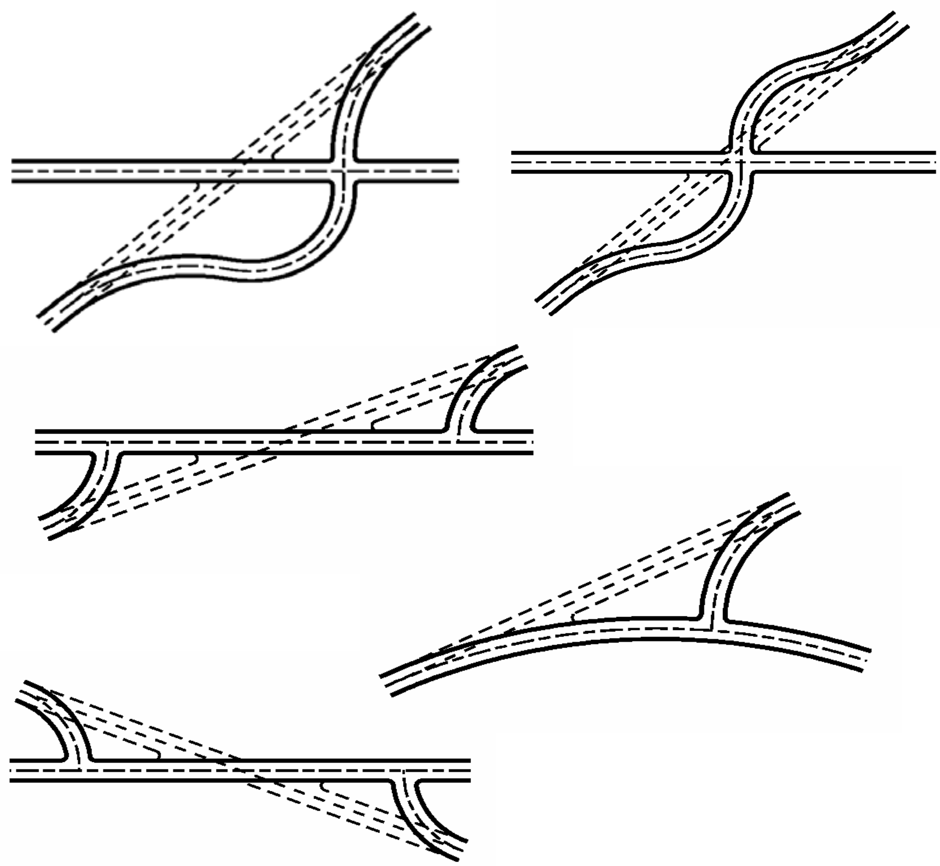
The angle of intersection of two highways can greatly influence the intersection's safety and operational characteristics. Both individual vehicle operations and the nature of vehicle/vehicle conflicts are affected by angle of intersection. Heavy "skew" intersection angles produce large open pavement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

1. Vehicles crossing the intersection are exposed for a longer time to conflicts from crossing traffic. This may be a particularly critical problem 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 motorists 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.

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]***. Realignments such as those shown in ***Figure 3-2*** should be made whenever feasible.

Figure 3-2 Intersection Realignment



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*S2)/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.

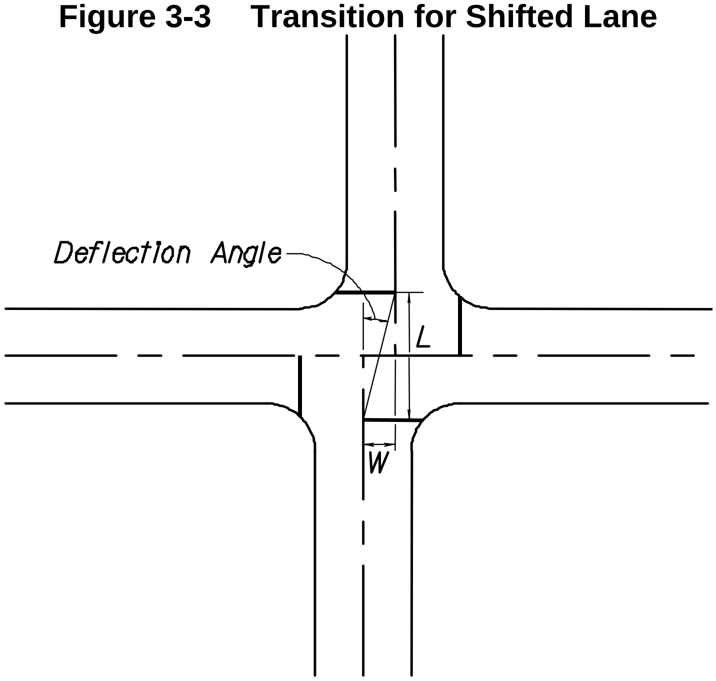
3.7.2.1 Transitions

The addition or deletion of through traffic lanes should desirably 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***.

3.7.2.2 Lane Shift

A lane shift through an intersection, should meet the general requirements for non-merging lanes. Short curves, however, may be desirable at each end, especially if pavement markings are used through the intersection to provide positive guidance to the motorist. The shifting taper length is controlled by the size of the intersection (see ***Figure 3-3***).

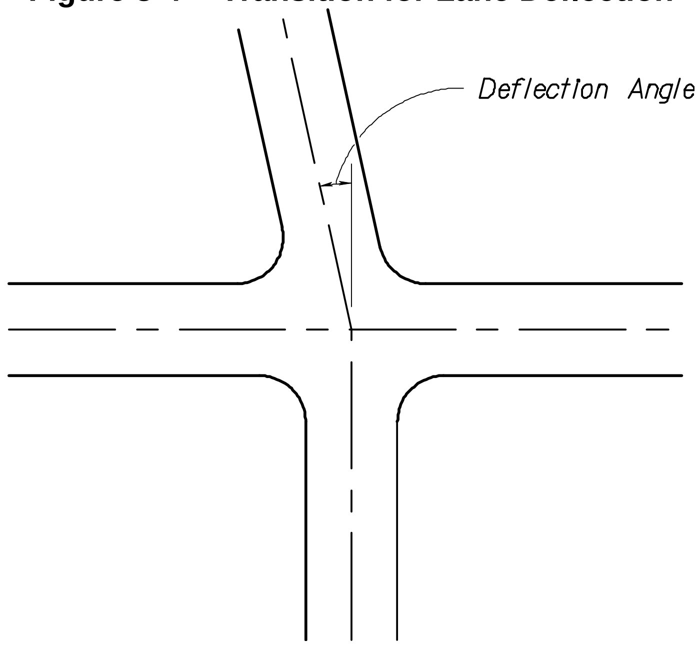
Figure 3-3 Transition for Shifted Lane



3.7.2.3 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



3.8 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, functional classification, drainage and terrain conditions. Within these basic controls, several general criteria must be considered, 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.

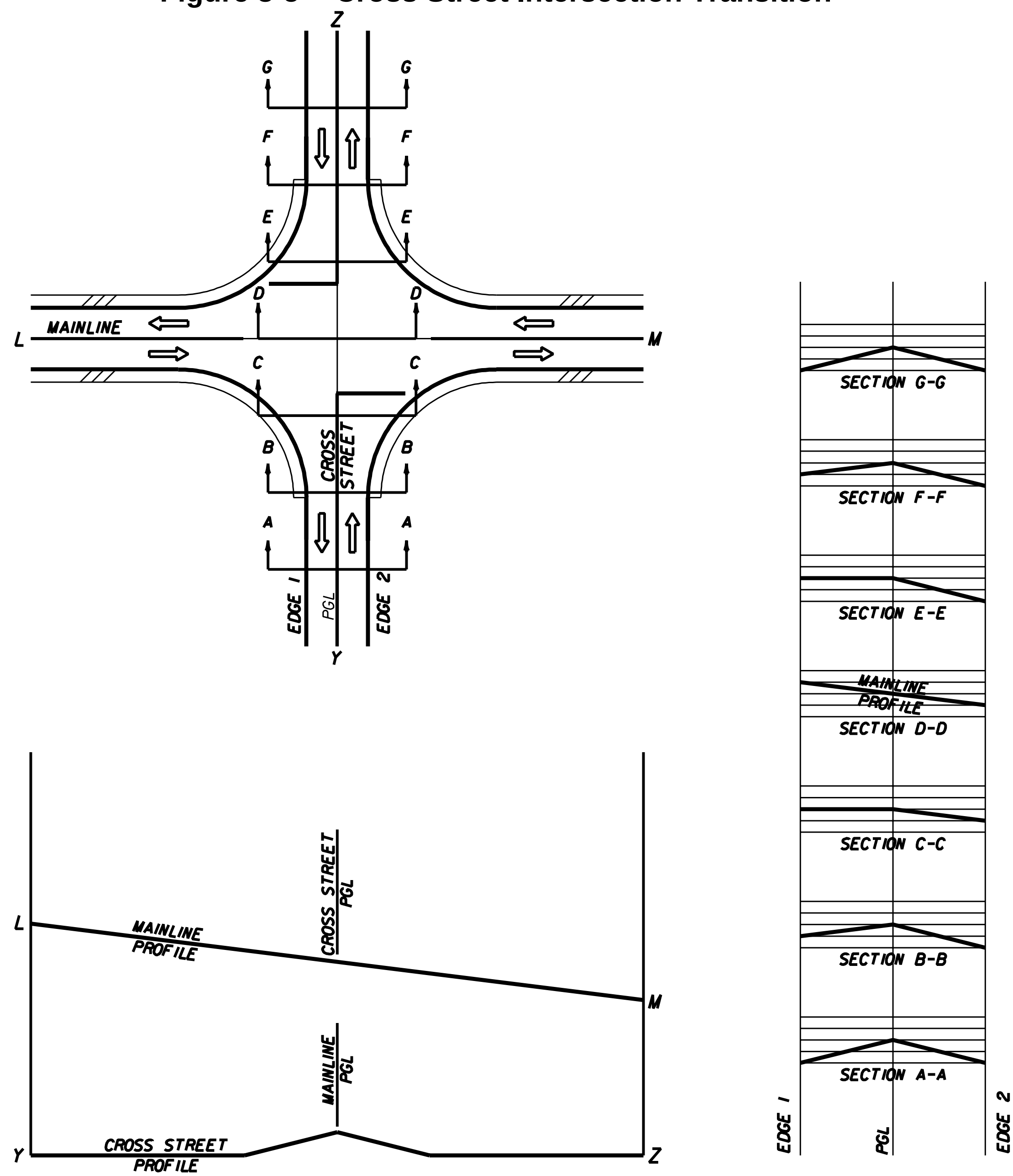
As a rule, the alignment and grades are subject to greater constraints at or near intersections than on the open road. Their combination at or near the intersection should produce traffic lanes that are clearly visible to drivers at all times and clearly understandable for any desired direction of travel,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

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 too expensive, grades should not exceed 6%.

The profile gradelines and cross sections on the intersection legs should be adjusted for a distance back from the intersection proper to provide a smooth junction and proper drainage. Normally, the gradeline 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***.

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 flowline at street intersections, profile gradeline, 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.

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 the transitioning of the roadway profiles and cross slopes at the approaches of an intersection.

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

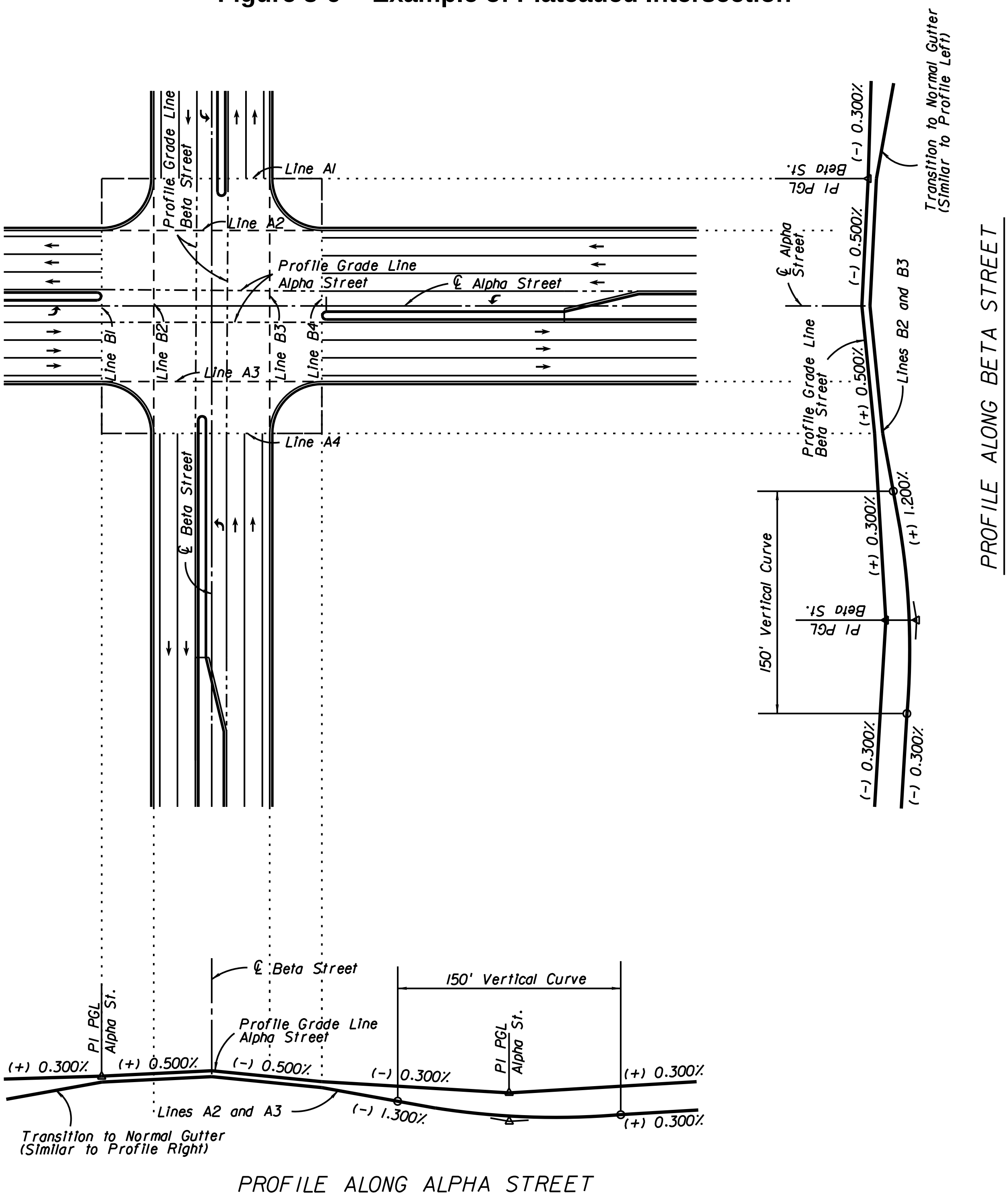
2. The profile combination selected and the cross section generated as a result should be sufficient to provide a smooth connection and proper drainage. On curb and gutter sections, care must be taken to ensure that the runoff-spread restrictions are not compromised.

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***. A step by step method is presented in ***Appendix A***.

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 3-4***.

Table 3-4 Minimum Lane Widths in Feet

Facility Type	Area Type	Through or Travel Lanes	Turn Lane (LT/RT/MED)	Bicycle Lane
Arterial	**Rural**	12	12	5_5
	Urban	12_1	$12_{1,4}$	4_6
Collector	**Rural**	12_7	$11_{2,4}$	5_5
	Urban	11_3	$11_{3,4}$	4_6
Local	**Rural**	10	10	5_5
	Urban	10	10	4_6

1. 11 ft. permitted on non-FIHS/SIS roads if one of these conditions exist:
   a. R/W and existing conditions are stringent controls
   b. Facility operates on interrupted flow conditions
   c. Design speed 40 mph or less
   d. Intersection capacity not adversely affected
   e. Truck volume 10% or less
2. 12 ft. lanes for all 2-lane rural.
3. 12 ft. lanes in industrial areas when R/W is available.
4. With severe R/W controls, 10 ft. turn lanes may be used where design speeds are 40 mph or less and the intersection is controlled by traffic signals. Median turn lanes shall not exceed 15 ft.
5. Designated or undesignated shoulder pavement.
6. Designated or undesignated.
7. 11ft. for low volume ADT

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. Auxiliary lanes are discussed further in ***Section 3.12***, turning roadway widths are covered in ***Section 3.14*** and other 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 3-5***.

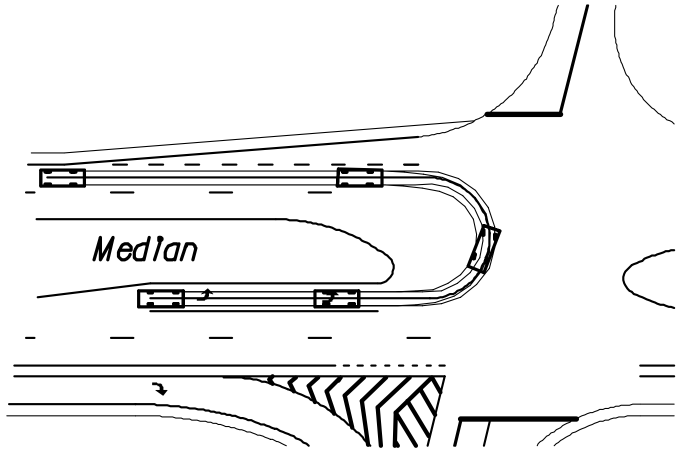
Wherever possible, a newly designed divided highway should have a median width that can accommodate normal left-turns and passenger car U-turns by using a sufficient intersection design and a median storage lane that will protect and store the design-hour turning volume (see ***Figure 3-7***). If adequate median width does not exist for accommodating U-turns, then adding extra pavement width, through use of a taper or on the shoulder for example, should be considered. Basic median functions and their required width are provided in ***Table 3-6***.

Table 3-5 Median Widths for Arterials and Collectors

Design Speed (mph)	Median Widths (feet)
> 45	40
≤ 45	22_1
Paved and Painted for Left Turns	12_2

1. 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 feet for design speeds = 45 mph and to 15.5 feet for design speeds ≤ 40 mph.
2. 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 ***PPM, Section 2.2.2*** and the ***Access Management Rules***.

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



Design Vehicle Passenger Car

Table 3-6 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

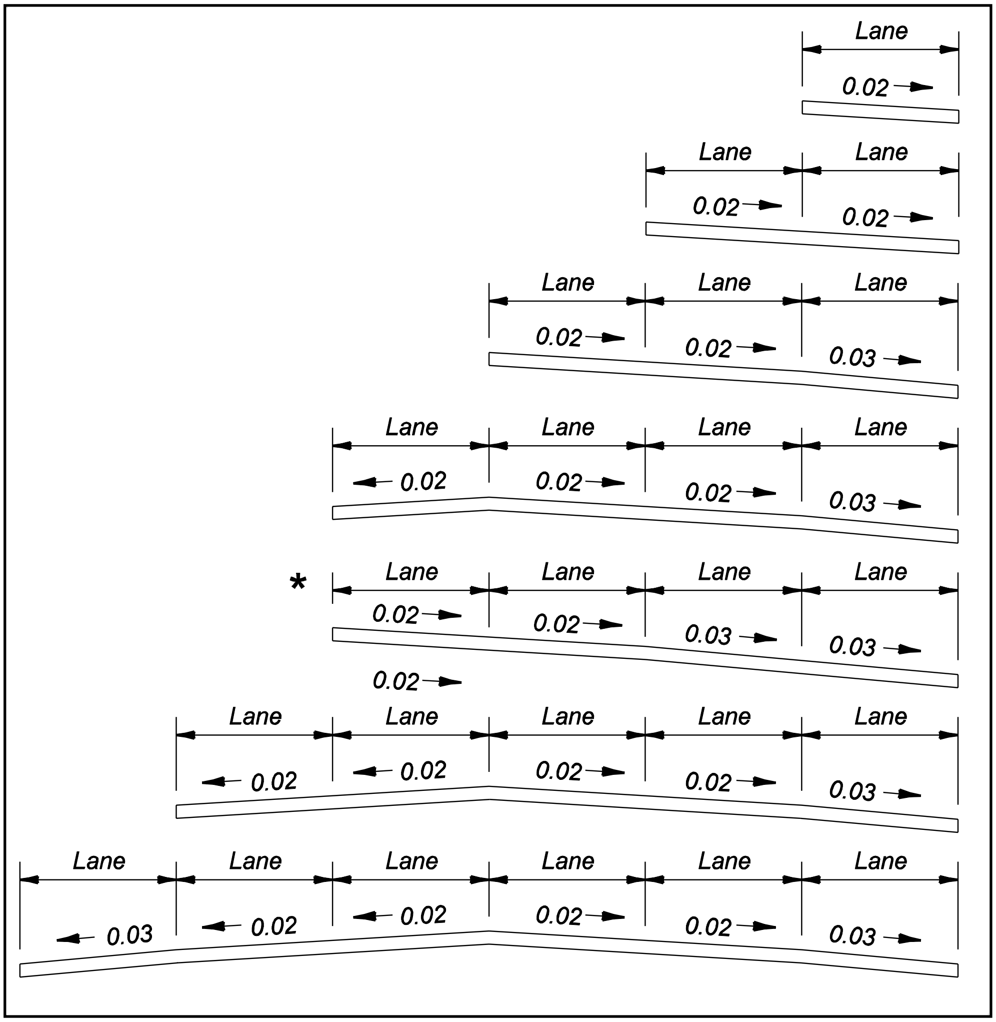
1. See Table 3-5 for conditions.
2. Based on the passenger car (P) design vehicle.

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. 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



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:

1. 0.04 for design speeds of 45 mph or less.
2. 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.

When right of way is restricted, shoulder widths for turning roadways and intersection returns with flush shoulders may not be reduced to less than 6 feet.

3.9.5 Curbs

Curbs are generally designed with a gutter to form a combination curb and gutter section. They are used to 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 objective 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 only used 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.

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 3-7***.

Figure 3-9 Border Width of Highways with Flush Shoulders

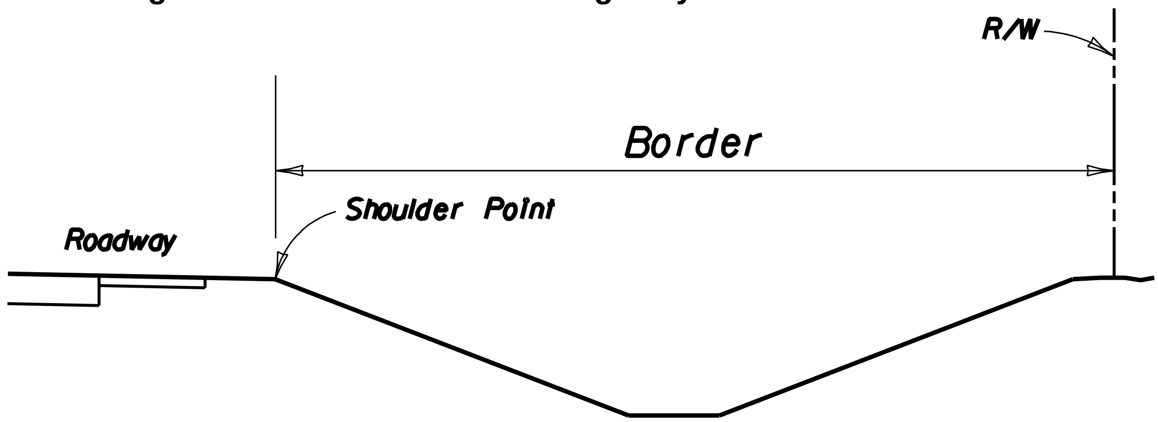


Table 3-7 Minimum Border Widths for Highways with Flush Shoulders

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

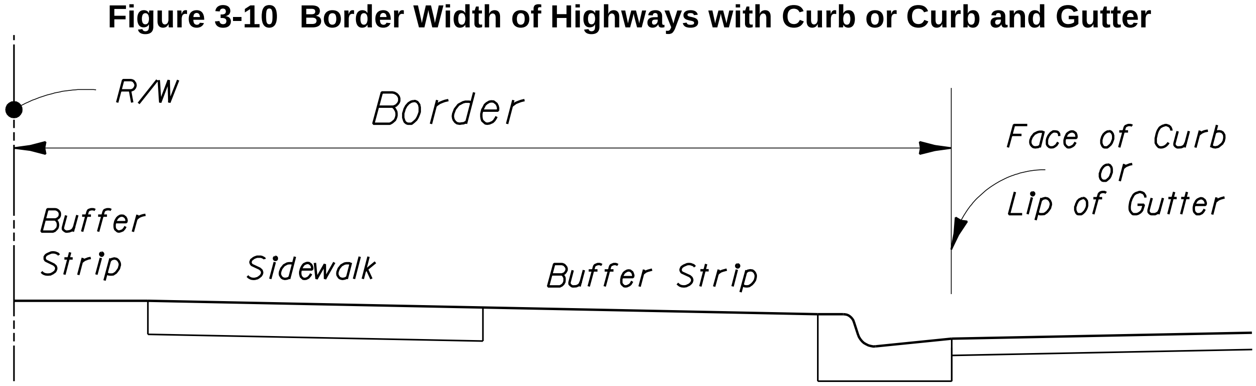


Table 3-8 Minimum Border Widths for Highways with Curbs or Curb and Gutter

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.

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 arterial roadways that do not have shoulders. 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 the sidewalk is adjacent to the curb, the minimum width of the sidewalk shall be 6 feet. Sidewalks adjacent to roadways with flush shoulders shall have a minimum sidewalk 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 shall allow for adequate drainage, but shall be no more than 2% to comply with ADA requirements.

3.10.4 Curb Ramps

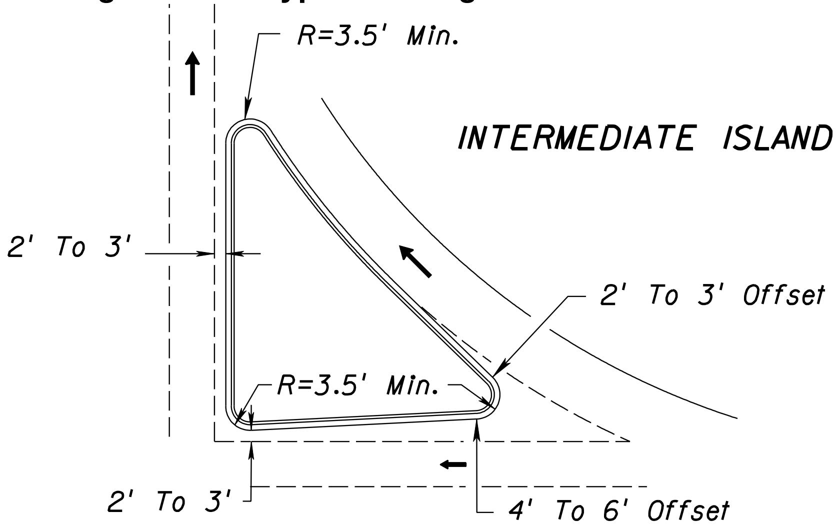
Pedestrian facilities must be designed in accordance with ADA to accommodate the physically and visually challenged citizens whose mobility is dependant on wheelchairs and other devices. In areas with sidewalks, curb ramps shall be incorporated at locations where a marked crosswalk adjoins the sidewalk.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

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.

3.11 Channalizing 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, provides maximum convenience and instills driver confidence. Improper channelization has the opposite effect and may be worse than no channelization. The general guidelines for channelization design include:

1. Conflicts should be separated so that drivers and pedestrians may deal with only one conflict and make only one decision at a time.
2. The proper traffic channels should seem natural and convenient to drivers and pedestrians.
3. The number of islands should be held to a practical minimum to avoid confusion.
4. Islands should meet minimum size requirements.
5. Channelization must be visible.
6. All movements must be accounted for.
7. The major traffic flows should be favored.
8. Islands should be designed for the design speed.
9. 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 are those with side dimensions of at least 100 feet [***AASHTO 2004, p. 722***].

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.

3.11.2 Island Delineation

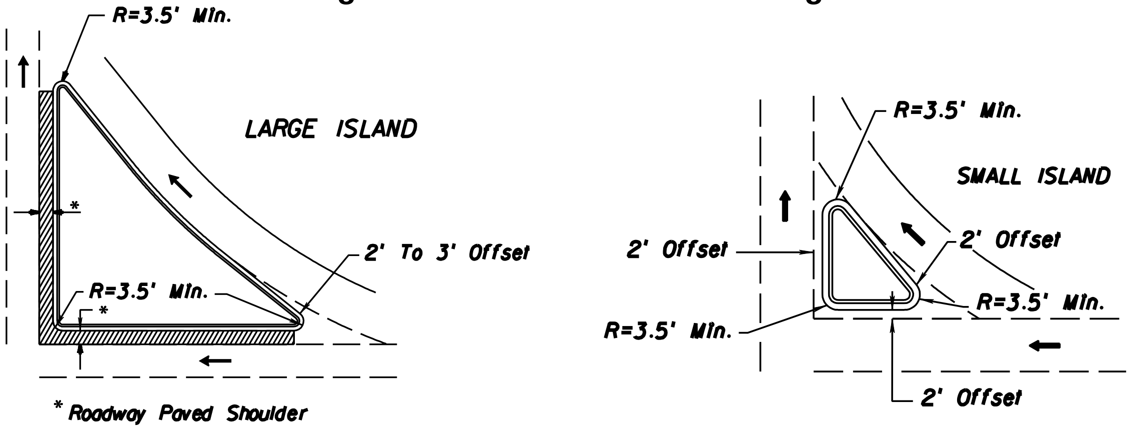
Islands should be delineated or outlined by a variety of treatments, depending on their size, location and function. Island delineation can be divided into three types: (1) raised-curb islands , (2) islands delineated by pavement markings or reflectorized markers placed on paved areas and (3) islands formed by the pavement edges, possibly supplemented by delineators on posts or other 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. 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, page 76***]:

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 in 3-second driving time to the intersection.
4. The width of the island should adequately serve its intended functions, as given previously in ***Table 3-6***.
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 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.

The general principles for 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 and preceding and following right-turning movements. The 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 3-4***.

The length of the auxiliary lanes consists of three components: (1) deceleration length, (2) storage 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 3-9*** and ***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 half of that required for single-lane operation.

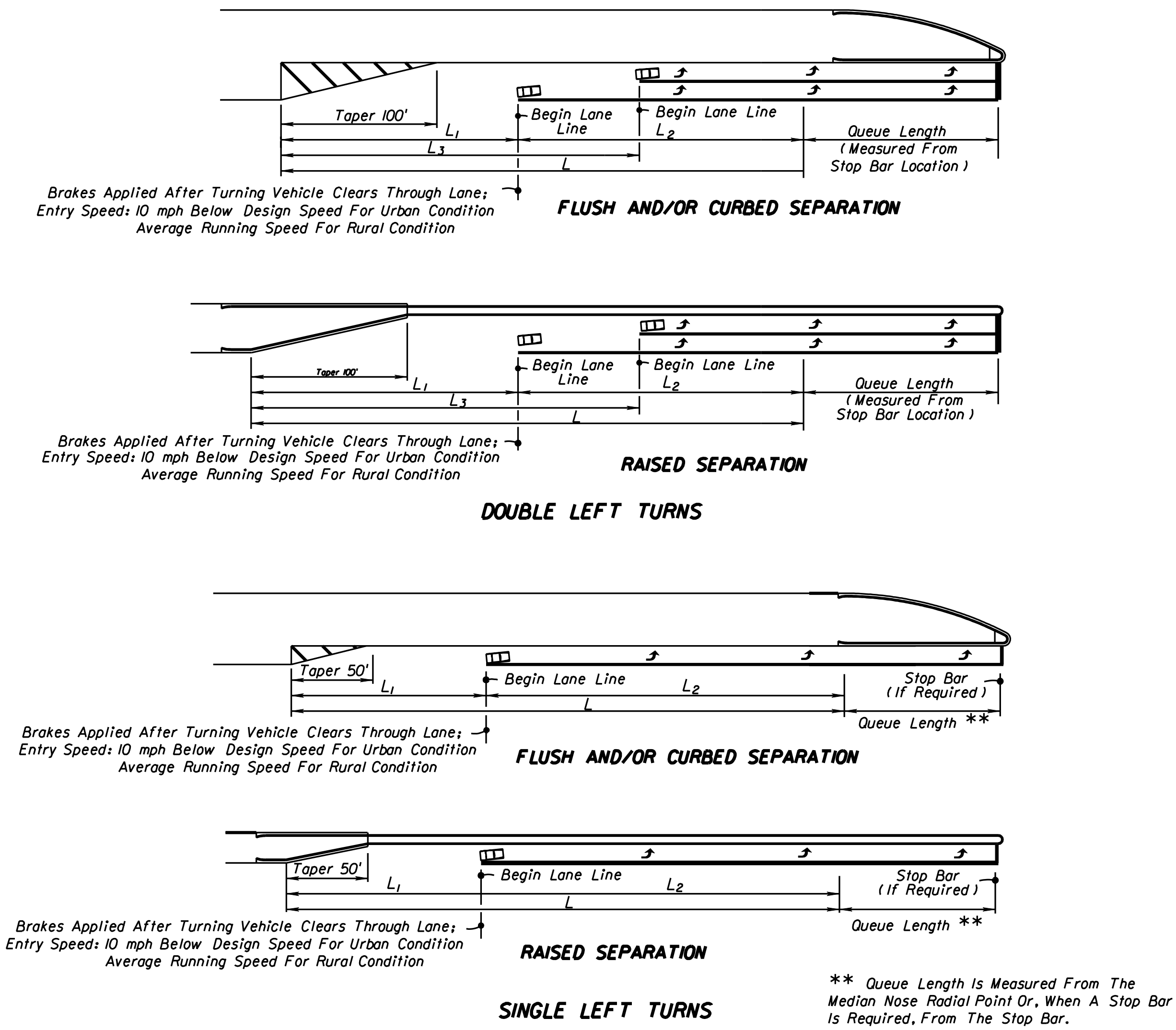
3.12.3 Approach End Taper

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 L1 and L3 account for the full transition lengths a road user will use to enter the auxiliary lane for various speed conditions assumed for design.

Table 3-9 Minimum Deceleration Lengths

Turn Lanes - Curbed and Uncurbed Medians								
			Urban Conditions			Rural Conditions		
Design Speed (mph)	**Entry Speed (mph)**	**Clearance Distance L1 (feet)**	**Brake To Stop Distance L2 (feet)**	**Total Decel. Distance L (feet)**	**Clearance Distance L3 (feet)**	**Brake To Stop Distance L2 (feet)**	**Total Decel. Distance L (feet)**	**Clearance Distance L3 (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



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 3-10*** and ***Table 3-11*** 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. 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

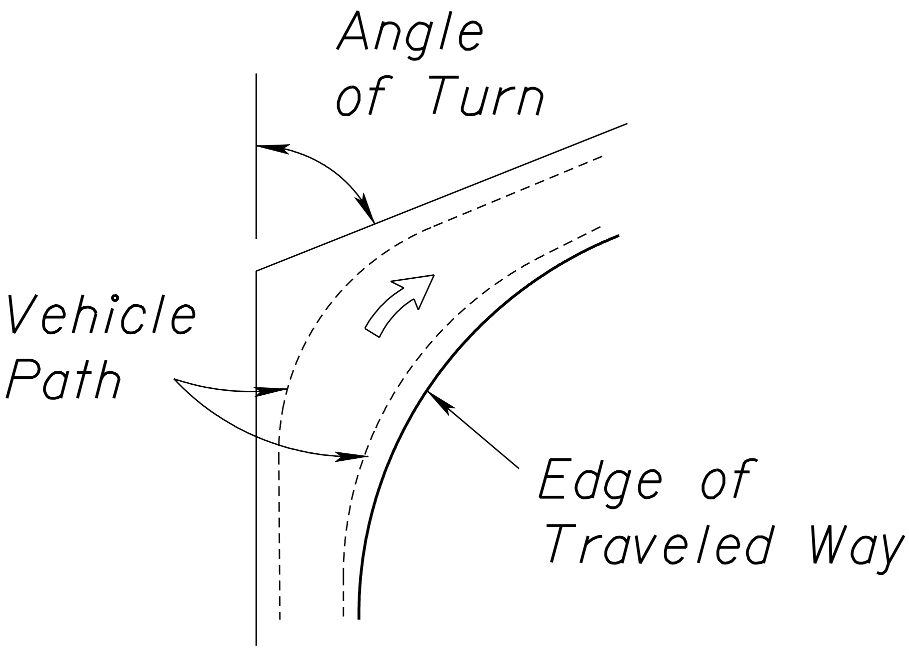


Table 3-10 Edge-of-Traveled-Way Designs for Turns at Intersections

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	100	----	----	----
	WB-40	150	----	----	----
	WB-50	200	----	----	----
	WB-62	360	220	3.0	15:1
	WB-67	380	220	3.0	15:1
	WB-100T	260	125	3.0	15:1
	WB-109D	475	260	3.5	20:1
45	P	50	----	----	----
	SU	75	----	----	----
	WB-40	120	----	----	----
	WB-50	175	120	2.0	15:1
	WB-62	230	145	4.0	15:1
	WB-67	250	145	4.5	15:1
	WB-100T	200	115	2.5	15:1
	WB-109D	----	200	4.5	20:1
60	P	40	----	----	----
	SU	60	----	----	----
	WB-40	90	----	----	----
	WB-50	150	120	3.0	15:1
	WB-62	170	140	4.0	15:1
	WB-67	200	140	4.5	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	55	45	2.0	10:1
	WB-40	----	60	2.0	15:1
	WB-50	----	65	3.0	15:1
	WB-62	----	145	4.0	20:1
	WB-67	----	145	4.5	20:1
	WB-100T	----	85	3.0	15:1
	WB-109D	----	140	5.5	20:1
90	P	30	20	2.5	10:1
	SU	50	40	2.0	10:1
	WB-40	----	45	4.0	10:1
	WB-50	----	60	4.0	15:1
	WB-62	----	120	4.5	30:1
	WB-67	----	125	4.5	30:1
	WB-100T	----	85	2.5	15:1
	WB-109D	----	115	2.9	15:1

Table 3-10 Continued

Angle of turn (degrees)	Design vehicle	Simple curve radius (feet)	Simple curve radius with taper		
			Radius (feet)	Offset (feet)	Taper H:V
105	P	----	20	2.5	8:1
	SU	----	35	3.0	10:1
	WB-40	----	40	4.0	10:1
	WB-50	----	55	4.0	15:1
	WB-62	----	115	3.0	15:1
	WB-67	----	115	3.0	15:1
	WB-100T	----	75	3.0	15:1
	WB-109D	----	90	9.2	20:1
120	P	----	20	2.0	10:1
	SU	----	30	3.0	10:1
	WB-40	----	35	5.0	8:1
	WB-50	----	45	4.0	15:1
	WB-62	----	100	5.0	15:1
	WB-67	----	105	5.2	15:1
	WB-100T	----	65	3.5	15:1
	WB-109D	----	85	9.2	20:1
135	P	----	20	1.5	10:1
	SU	----	30	4.0	10:1
	WB-40	----	30	8.0	15:1
	WB-50	----	40	6.0	15:1
	WB-62	----	80	5.0	20:1
	WB-67	----	85	5.2	20:1
	WB-100T	----	65	5.5	15:1
	WB-109D	----	85	8.5	20:1
150	P	----	18	2.0	10:1
	SU	----	30	4.0	8:1
	WB-40	----	30	6.0	8:1
	WB-50	----	35	7.0	6:1
	WB-62	----	60	10.0	10:1
	WB-67	----	65	10.2	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	1.5	10:1
	WB-40	----	20	9.5	5:1
	WB-50	----	25	9.5	5:1
	WB-62	----	55	10.0	15:1
	WB-67	----	55	13.8	10:1
	WB-100T	----	55	10.2	10:1
	WB-109D	----	55	20.0	10:1

Table 3-11 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	----	----	----	----
	WB-40	----	----	----	----
	WB-50	----	----	----	----
	WB-62	----	----	----	----
	WB-67	460-175-460	4.0	300-175-550	2.0-4.5
	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	----	----	----	----
	WB-40	----	----	----	----
	WB-50	200-100-200	3.0	----	----
	WB-62	460-240-460	2.0	120-140-500	3.0-8.5
	WB-67	460-175-460	4.0	250-125-600	1.0-6.0
	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	----	----	----	----
	WB-40	----	----	----	----
	WB-50	200-75-200	5.5	200-75-275	2.0-7.0
	WB-62	400-100-400	15.0	110-100-220	10.0-12.5
	WB-67	400-100-400	8.0	250-125-600	1.0-6.0
	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	120-45-120	2.0	----	----
	WB-40	120-45-120	5.0	120-45-195	2.0-6.5
	WB-50	150-50-150	6.5	150-50-225	2.0-10.0
	WB-62	440-75-440	15.0	140-100-540	5.0-12.0
	WB-67	420-75-420	10.0	200-80-600	1.0-10.0
	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	120-40-120	2.0	----	----
	WB-40	120-40-120	5.0	120-40-200	2.0-6.5
	WB-50	180-60-180	6.5	120-40-200	2.0-10.0
	WB-62	400-70-400	10.0	160-70-360	6.0-10.0
	WB-67	440-65-440	10.0	200-70-600	1.0-11.0
	WB-100T	250-70-250	4.5	200-70-300	1.0-5.0
	WB-109D	700-110-700	6.5	100-95-550	2.0-11.5

Table 3-11 Continued

Angle of turn (degrees)	Design vehicle	3-Centered compound			
		Curve radii (feet)	Symmetric offset (feet)	Curve radii (feet)	Asymmetric (feet)
105	P	100-20-100	2.5	----	----
	SU	100-35-100	3.0	----	----
	WB-40	100-35-100	5.0	100-55-200	2.0-8.0
	WB-50	180-45-180	8.0	150-40-210	2.0-10.0
	WB-62	520-50-520	15.0	360-75-600	4.0-10.5
	WB-67	500-50-500	13.0	200-65-600	1.0-11.0
	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	100-30-100	3.0	----	----
	WB-40	120-30-120	6.0	100-30-180	2.0-9.0
	WB-50	180-40-180	8.5	150-35-220	2.0-12.0
	WB-62	520-70-520	10.0	80-55-520	24.0-17.0
	WB-67	550-45-550	15.0	200-60-600	2.0-12.5
	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	100-30-100	4.0	----	----
	WB-40	120-30-120	6.5	100-25-180	3.0-13.0
	WB-50	160-35-160	9.0	130-30-185	3.0-14.0
	WB-62	600-60-600	12.0	100-60-640	14.0-7.0
	WB-67	550-45-550	16.0	200-60-600	2.0-12.5
	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	100-30-100	4.0	----	----
	WB-40	100-30-100	6.0	90-25-160	1.0-12.0
	WB-50	160-35-160	7.0	120-30-180	3.0-14.0
	WB-62	480-55-480	15.0	140-60-560	8.0-10.0
	WB-67	550-45-550	19.0	200-55-600	7.0-16.4
	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
180	P	50-15-50	0.5	----	----
	SU	100-30-100	1.5	----	----
	WB-40	100-20-100	9.5	85-20-150	6.0-13.0
	WB-50	130-25-130	9.5	100-25-180	6.0-13.0
	WB-62	800-45-800	20.0	100-55-900	15.0-15.0
	WB-67	600-45-600	20.5	100-55-400	6.0-15.0
	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 3-12 summarizes the operational characteristics of various corner radii for the range of design vehicles.

Table 3-12 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 or WB-50 vehicle with minor encroachment
50	Moderate speed turns for all vehicles up to WB-50

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

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.

Within an intersection, road users anticipate the sharp curves and accept operation with higher side friction than they accept 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 3-13*** 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 3-13 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: Preferably 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.

The design widths for turning roadways for different design vehicles for each of the above cases are given in ***Table 3-14***.

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 3-14 Design Widths for Turning Roadways [AASHTO 2004]

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 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 ***Exhibit 3-51, AASHTO 2004***.

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 3-15***.

Figure 3-15 Maximum Algebraic Difference at Turning Roadway Terminals

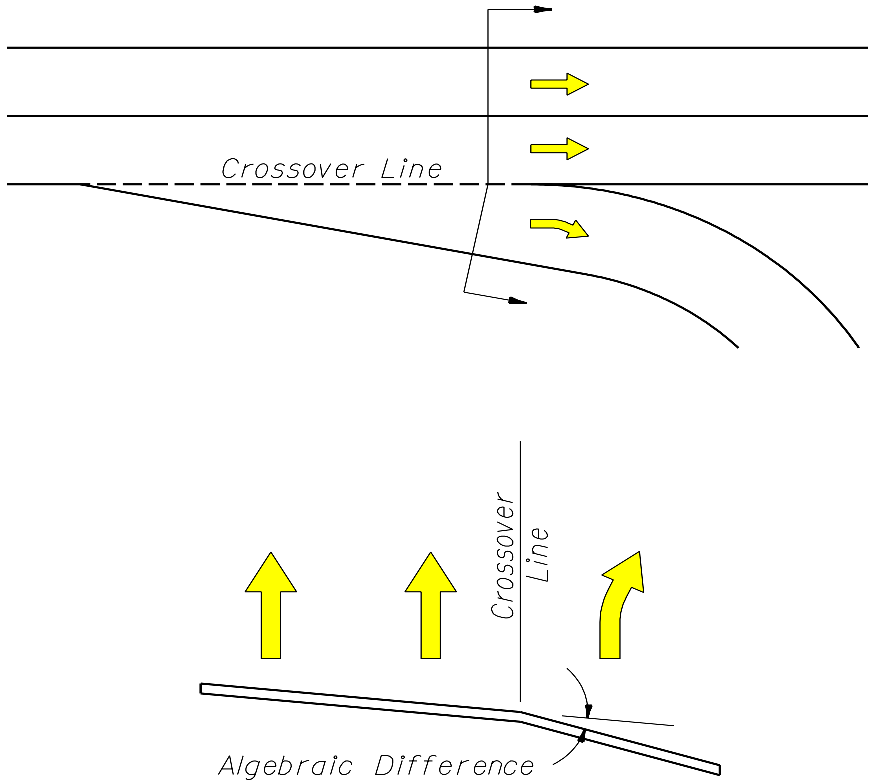


Table 3-15 Maximum Algebraic Difference in Cross Slope at Turning Roadway Terminals

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 3-16*** provides control radii for minimum speed turns (10 to 15 mph) that can be used for establishing the location of the median ends.

Table 3-16 Control Radii for Minimum Speed Turns

Design Vehicles Accommodated	**Control Radius (feet)**		
	50 (40 min)	60 (50 min)	75
Predominant	P	SU	WB-40
Occasional	SU	WB-40	WB-50

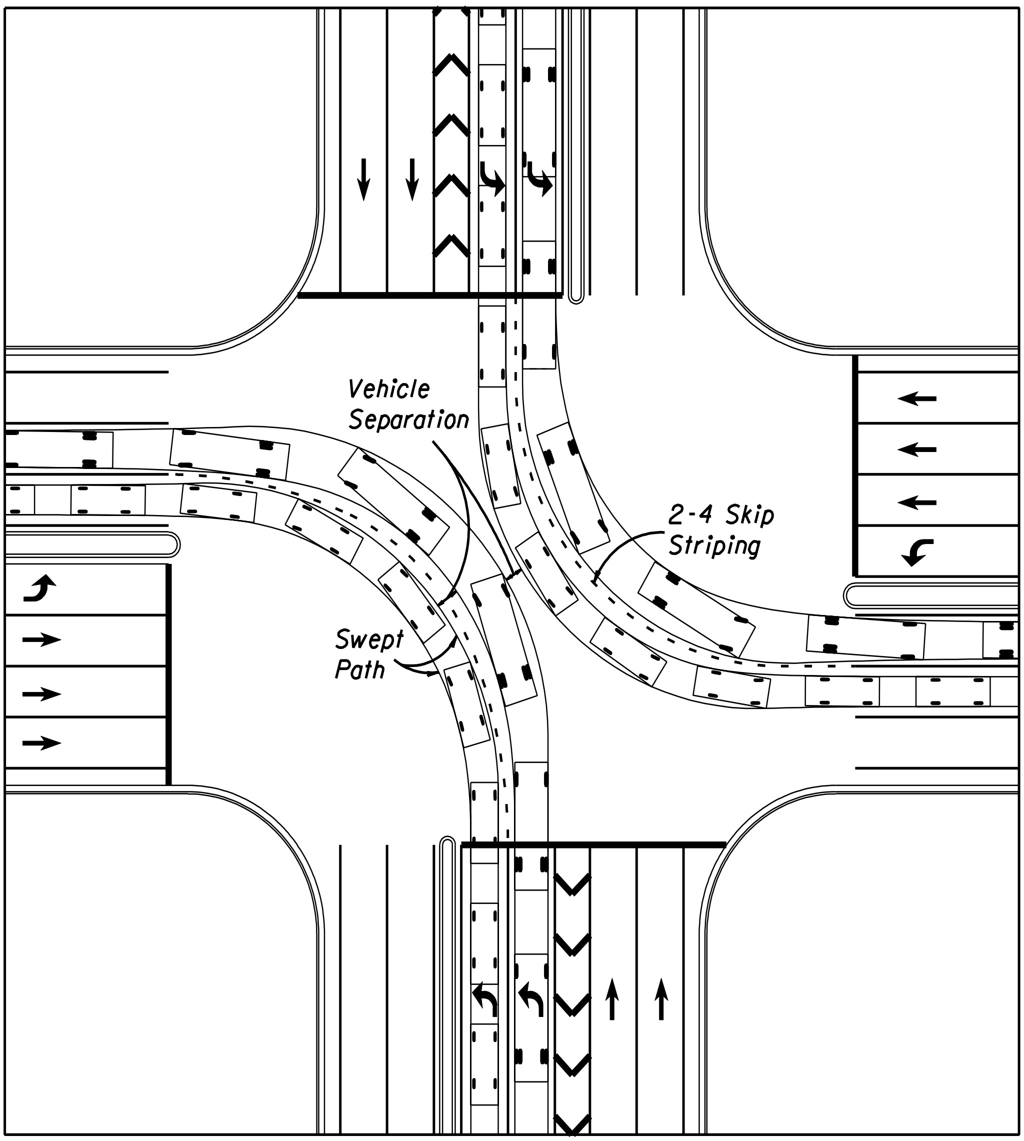
3.13.7 Double Left and Double Right Turning Lanes

When double left turn or double right turn lanes are provided, special consideration must be given to providing turning radii to accommodate vehicles turning two 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 lane turns should consider as a minimum an SU vehicle and P vehicle turning simultaneously as shown in ***Figure 3-16***.

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 Intersection Sight Distance

Inadequate sight distance is a contributing factor in a large percentage of intersection crashes. The intersection design must provide sight distances 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 limited by the roadway geometry and the nature and development of the area adjacent to the roadway.

Intersection Sight Distance requirements provided in this document are based on the ***2004 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	Intersections with no control
Case B	Intersections with 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	Intersections with 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	Intersections with traffic signal control
Case E	Intersection with all-way stop control
Case F	Intersections with 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 either stop or signal controlled. In other words, a vehicle approaching an at-grade intersection on the SHS is assumed to have to stop before entering the intersection. Consequently, Cases A and C 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 (for both Cases A and C). These distances are given in ***Table 3-17***. 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 3-17 Minimum Stopping Sight Distance

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

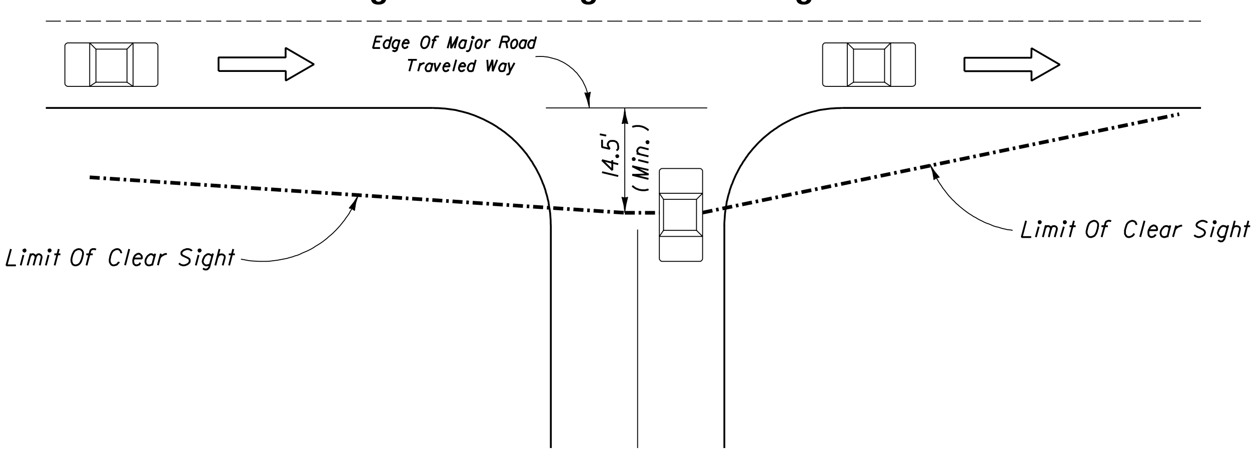
ADJUSTMENT IN DISTANCE FOR GRADES GREATER THAN 2%

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, which 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 3-18***). 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). These values also accommodate the Case F scenario.

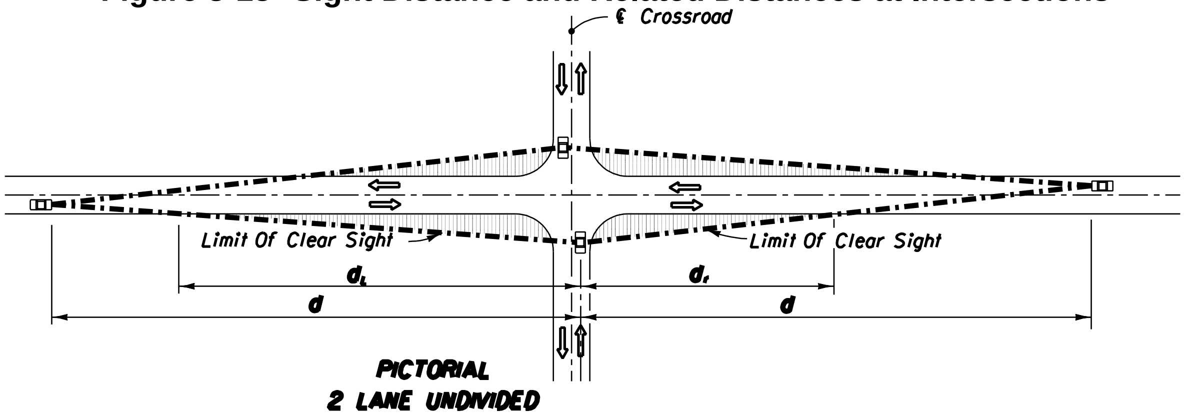
Sight distance (d) is measured along the major roadway from the center of the intersecting roadway. It applies to normal and skewed intersections (intersecting angles between 60° and 120°) and where vertical and/or horizontal curves are present. Related distances dL and dr are measured from the centerline 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.

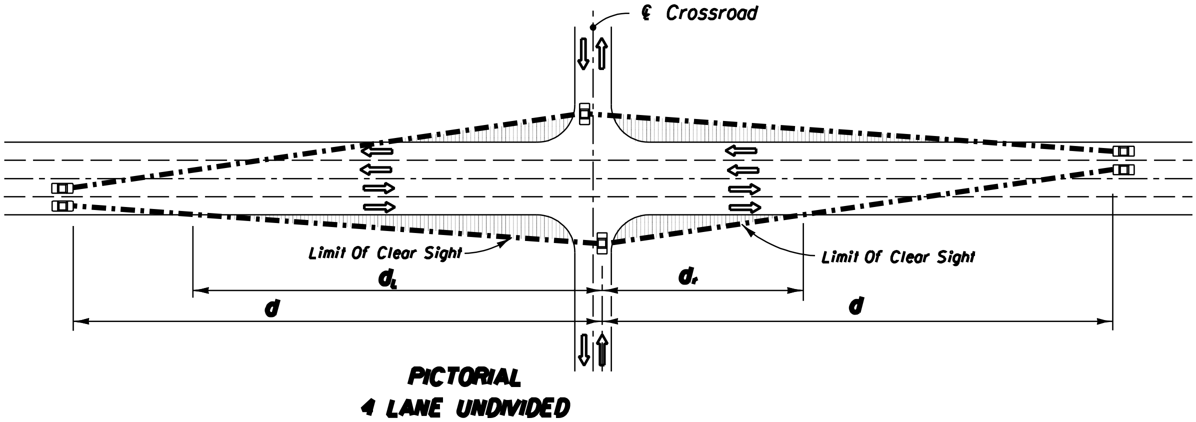
Table 3-18 Minimum Sight Distance and Related Distances for 2-Lane Undivided, Multilane Undivided and Multilane Divided Roadways

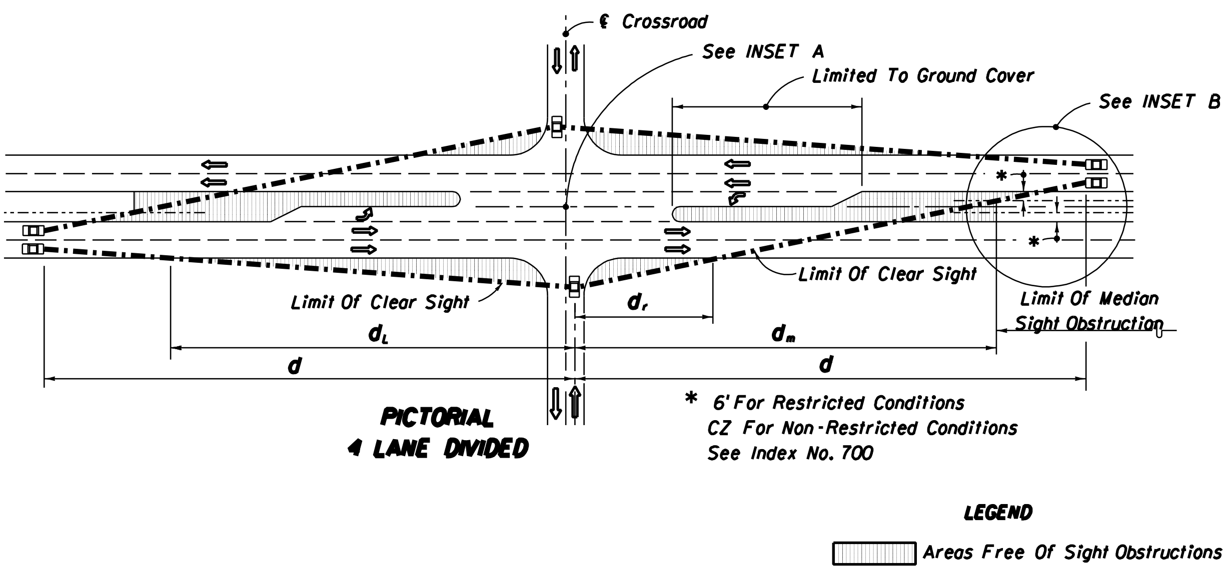
Type Facility	Distance (feet)	Design Speed (mph)							
		30	**35**	**40**	**45**	**50**	**55**	**60**	**65**
Two-lane Undivided	d	335	390	445	500	555	610	665	720
	dL	240	275	315	350	390	430	470	510
	dr	150	175	200	225	250	275	300	325
Multilane Undivided	d	355	415	475	530	590	650	705	765
	dL	250	295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

NOTE: Additional tables are available in ***Design Standards, Index 546*** that addresses 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.

3.14.4 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 may rearrange, relocate or eliminate 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 3-19***) 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 3-19 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		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.

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.

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Chapter 4

Signalization

Figures

Tables

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Chapter 4

Signalization

4.0 General

The 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 ***MUTCD*** establishes nationwide standards that promote uniformity to facilitate driver comprehension of traffic control devices. ***Part 4*** of the ***MUTCD*** covers 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.

MUTCD Part 4 Sections

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

2. The ***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 2000***.

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

   A. Certification and specialty items,

B. Stop line location,

C. Controller assemblies,

D. Left turn treatments

E. Signal preemption,

F. Lane configuration,

G. Signal Loops and

H. Mast arm supports.

Many of the same topics are also covered in this chapter. The guidelines presented here are consistent with the requirements of the ***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 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 "B" & "C"***
Index 17745	***Mast Arm Assemblies***
Index 17746	***Monotube Signal Structure***
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.***

The last index 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. Signal warrant studies,

B. Roundabout justification studies,

C. Vehicular and pedestrian counts and characteristics,

   D. Collision and condition diagrams,
   E. Performance evaluation studies and
   F. Traffic and pedestrian flow characteristics.

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. Mast arm supports,
   G. Computer models for traffic engineering and ITS analysis and design and
   H. Certification and approval of traffic control devices.

8. ***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:

10. ***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 is an emerging standard that incorporates contemporary computer and communications technology.

11. ***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.

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

13. 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 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}***.

4.2.1 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 and
4. Vehicle detectors, which may require parameter settings.

The typical positioning of the controller, detectors and conflict monitor is illustrated in ***Figure 4-1***.

Figure 4-1 Controller, Detectors and Conflict Monitor in a Cabinet

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 beyond the scope of this document.

4.2.2 Design Products and Prerequisites

The products of the signalization design process are generally a plan 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 signalization for each intersection should be individually designed ***{PPM}***.

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:

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.

4.2.3 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.

> Pavement markings should be used at traffic control signal locations. If the road surface will not retain pavement markings, signs should be installed to provide the needed road user information ***{MUTCD}***.

The above list is presented more or less in the 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.

4.3 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 ***{MUTCD}***.

4.3.1 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}***.

4.3.2 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 vehicular signal indications are not visible to pedestrians using crosswalk
> 4. At established school crossings at any signalized location ***{MUTCD}***.

1. When pedestrians are expected to cross different parts of the street on different phases;
2. When protected left turn phases are included in the signal sequence;
3. When the crash history indicates a hazard that could be mitigated by pedestrian signals;
4. When the crosswalk is a part of an established pedestrian or bicycle corridor;
5. When the crosswalk is used by people with special needs or
6. 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}***

4.3.3 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. 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 combined with 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 direction. 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.

4.4 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.

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 4.14*** 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.

> At least one signal face shall be located between 40 and 150 ft. from the stop line ***{MUTCD}***.

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 districts. It is generally agreed that mast arms that are used for signal

> Pavement markings should be used at traffic control signal locations. If the road surface will not retain pavement markings, signs should be installed to provide the needed road user information ***{MUTCD}***.

mounting should be parallel to the stop bar. This consideration can affect both the signal mounting method and the stop line location.

4.5 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.

4.5.1 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 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.

4.5.2 Need for Exclusive Left Turn Lanes

Left turns may be made from shared lanes 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.

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 should normally be provided with exclusive left turn lanes unless left turning volumes are negligible or unless the addition of a left turn lane would cause problems for 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 problems 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 several 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. Pedestrian signals should always be used for any crosswalk in which pedestrians will encounter protected left turns, regardless of the number of lanes.

4.5.3 Need for Left Turn Protection

There are no numerical warrants that have been 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 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).

Capacity analysis is a useful tool, however, safety concerns are generally raised by the public and sometimes by the crash records themselves at volume levels well below the capacity of the left-turning movement. The crash history 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.

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

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.

4.5.4 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.

Conditions Requiring Protected-only Left Turn Treatment

Turn lanes that are separated from the through lanes by more than 12 feet by a raised or painted island shall not be operated in the permissive mode. ***{PPM}***

Permissive movements should not be allowed for dual turn lanes. ***{PPM}***

Sight distance is less than 250 feet or 400 feet, for opposing traffic speeds less than or greater than 40 MPH, respectively. ***{TEM}***

The approach is the lead portion of a lead/lag phasing sequence. ***{TEM}***

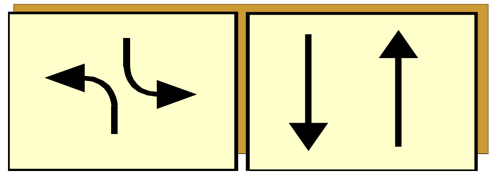
Protected-only phasing should be used at all intersections where protected-permitted phasing is likely to present a hazard to left turning traffic or pedestrians that conflict with left turning traffic. There are specific conditions that explicitly preclude left turns on permitted phases as indicated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

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 accidents per year on an approach,
4. The intersection configuration is 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 that present a greater challenge to the reaction times of motorists who are expected to yield to them.

4.5.5 Leading vs. Lagging Left Turns

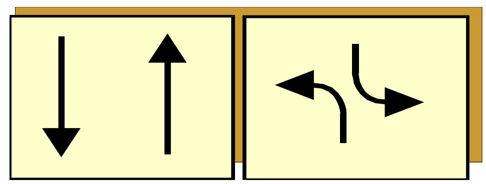
Protected left turns that precede the through movements are referred to as leading left turns. Those that follow the through movement are referred to as lagging left turns. The predominant practice in Florida strongly favors leading left turns. Lagging left turns may offer some benefit under the following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



Leading left turn protection

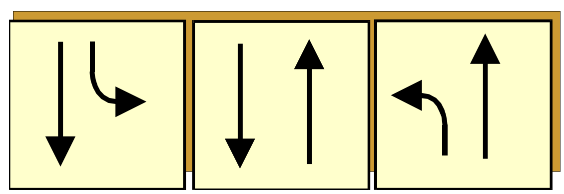
1. Modeling of arterial operations indicates that better progression may be achieved by a lagging left turn in one direction, often accompanied by a leading left turn in the other direction. This scheme is generally referred to as "lead-lag" phasing.

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.



Lead-lag phasing

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 ***[TEM]***:

1. "T" intersections and one-way streets where U turns are prohibited in the opposing direction;

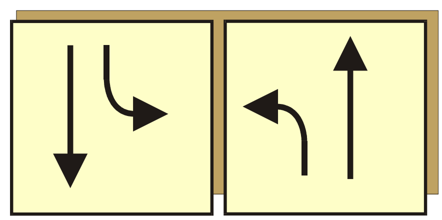
2. Four-way intersections where left turns are prohibited on the opposing approach or when the opposing approach has protected only left turn phasing.

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, including those that have no associated protected phase.

4.5.6 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:



Split-phase operation

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.

4.6 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.

4.6.1 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}***.

4.6.2 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 pedestrians. This requirement could be satisfied by installing a “U Turn must yield” sign facing the direction from which the U turn originates.

4.7 Determination of the Required Lengths for Turning Bays

Although many traffic models that provide queue length estimates have been developed, the definition, validity and applicability of their queue measures is not well understood. Some of the models are complex and theoretical, while others are more general and simplistic. The procedures used by these models are based on different queue definitions and have different computational approaches that lead to different results. A better understanding of queue length analysis procedures requires some additional explanation.

There are two very different queue definitions reported by different models, both of which are expressed in number of vehicles in the queue.

1. The queue accumulation refers to the number of vehicles actually contained in the queue at any point in time and
2. The queue reach refers to the position of the last vehicle in the queue.

The queue accumulation is important to the determination of delay and level of service, but the queue reach is the measure that determines the adequacy of the available storage. The main reason for the difference between these two definitions is that vehicles continue to join the back of the queue after the end of the red phase, as vehicles begin to depart from the front of the queue. The maximum queue accumulation is achieved at the end of the red phase. The maximum queue reach is not achieved until the instant before the queue has completely dissipated. The determination of the physical position of the last vehicle in the queue requires an assumption of average vehicle length, usually about 25 feet.

There are several variations of the above definitions used by the various models:

1. Uniform arrivals per cycle (UAC): The number of vehicles that would arrive on each cycle, assuming uniform arrival conditions;

2. Uniform maximum queue accumulation (UMQA): The number of vehicles in the queue at the beginning of green, assuming uniform arrival conditions;
3. Uniform maximum queue reach (UMQR): The maximum extent to which the queue backs up assuming uniform arrival conditions and
4. Adjusted maximum queue reach (AMQR): The maximum extent to which the queue backs up after performing an adjustment for randomness in the arrival conditions.

Only the last definition above reports an estimate of the queue size suitable for determining the storage lane requirements directly. All other definitions require an adjustment.

In an effort to standardize the methodology for queue length analysis, a UMQA-based queue length model has been introduced into the ***HCM 2000***. This model was developed by Akcelik as an adaptation of the method used in the SIDRA program. The heart of this method is a detailed procedure for estimating the queue length within confidence limits for any signalized movement.

The complexity of the procedure suggests that one of the available software products should be used for the necessary computations. Several queue length estimation models are incorporated into various signal timing design and analysis software products in common use throughout the state. A summary of the queue definition used by each of these products is presented in ***Table 4-1***.

The ***PPM, Volume I, Chapter 7*** suggests the use of the ***NCHRP 279*** method. This is the most approximate of all the models represented in ***Table 4-1***. Its use should be limited to situations in which computer software is not available. The NCHRP model is based on an average vehicle spacing of 25 feet and has the following formulation:

$$Q=(2.0)\,(DHV)\,(25)\,/\,N$$

where:

Q = design length for the left turn storage (ft).
DHV = left turn volume (vph) during design peak hour.
N = number of cycles per hour for peak hour (use N = 30 as default.)

Most of the models apply one or more adjustments to account for the effects of random and overflow phenomena on queue lengths. Some use a constant multiplier in the range of 2.0 to reflect the aggregate of all effects, while others are more detailed in their treatment. The direct application of the Poisson distribution suggests that 90% confidence in the adequacy of the storage space may be achieved with queue adjustment factors between 1.5 and 2.0, depending on the average queue length. Common wisdom has evolved from this process over the years to suggest that queue adjustment factors in this range are reasonable and appropriate.

It is important to note that the queue adjustment factors produced by the ***HCM 2000*** procedure suggest that the adjustment factor is sometimes considerably higher than the accepted value of 2.0 and may actually exceed 5.0 under very high degrees of saturation. The lesson here is that, while the currently accepted values may be reasonable at lower degrees of saturation, it is very difficult to achieve a design that will only fail on one cycle out of 10 when volumes are very close to capacity.

Table 4-1 Queue Definitions Used by Different Estimation Procedures

MODEL	REPORTED QUEUE DEFINITION
SIDRA	AMQR
SOAP	UMQR
TRANSYT-7F	UMQR
PASSER II	UMQR
SYNCRO	UMQR (Average & 90%)
NCHRP 279	UAC
NETSIM	Historical maximum queue accumulation
SIGNAL 97	UMQA
OPPENLANDER	UMQA

4.8 Selection of Control Equipment

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*** and of the FDOT ***Minimum Specifications for Traffic Control Signal Devices (MSTCSD)***.

The traffic signal equipment installed in Florida is required to be certified by the FDOT's Traffic Engineering Office in Central Office. Non-certified equipment cannot be used ***{PPM}***.

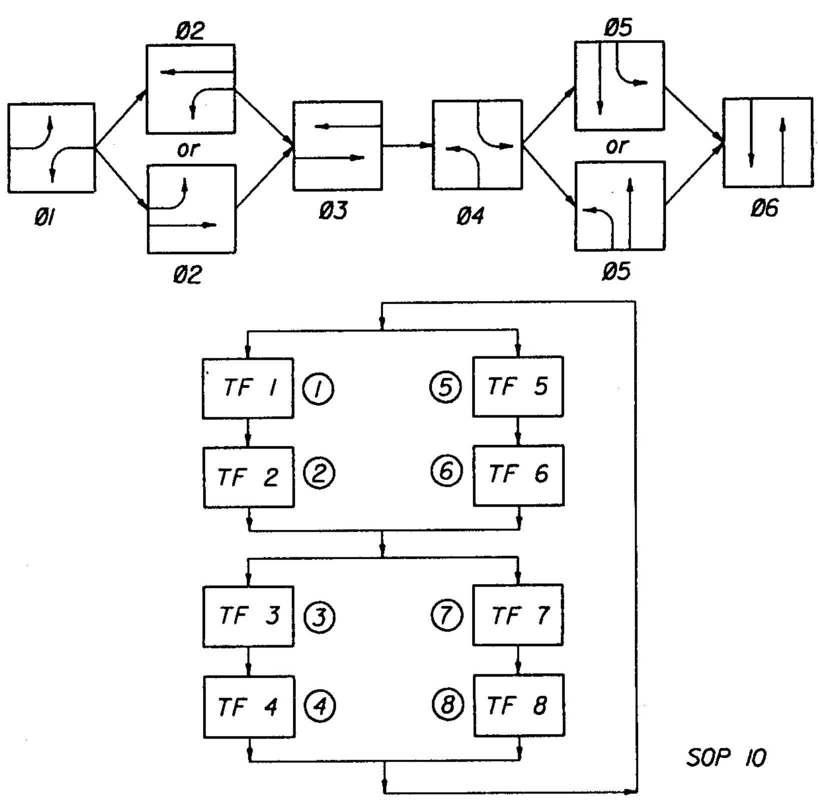
The review process involves an initial review based on documentation submitted by an applicant. The FDOT requires a subsequent review of the actual product, performed on hardware sample(s) furnished by the applicant. Products that are determined to meet the required criteria receive a certification number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

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

1. Pretimed Traffic Signal Controllers,
2. Traffic-Actuated Traffic Signal Controllers: solid state,
3. Solid-State Pretimed 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-1. Inductive loop detectors are the most common choice for vehicle detection. 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.

Figure 4-2 Example of a Standard Operating Plan (SOP) for Signal Phasing (Design Standards, Index 17870)



4.9 Selection of Phasing Plan

The phasing plan selection should be based on the standard eight-phase dual ring control scheme prescribed in ***Design Standards, Index 17870***, as shown in ***Figure 4-2***. The concepts of single-ring sequential and dual-ring concurrent phasing representation were discussed previously in Chapter 2. In the following discussion, it is assumed that the reader is familiar with these concepts.

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.

Engineering judgment should be used to determine the proper phasing and timing for a traffic control signal ***{MUTCD}***.

4.10 Assignments 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.

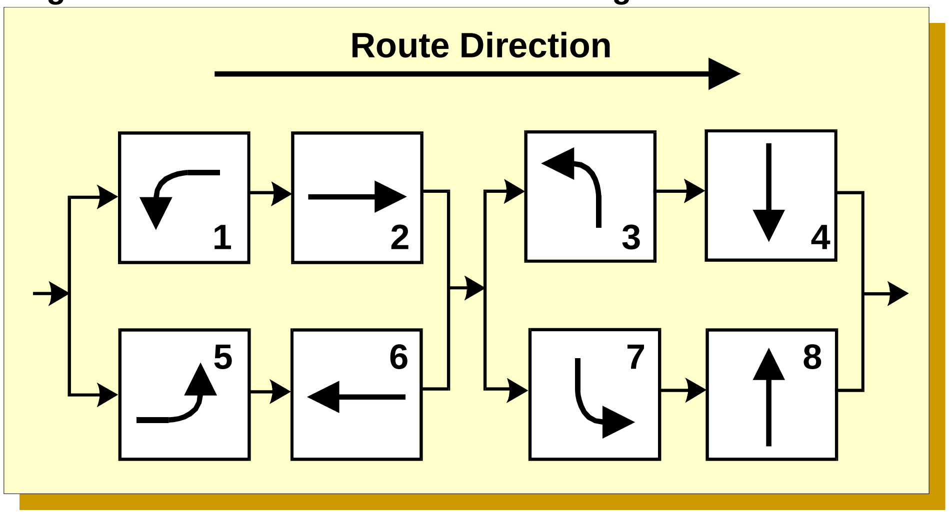
4.10.1 NEMA Conventions

The NEMA conventions offer some latitude in the assignment of 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 4-3***. The use of this convention will facilitate the transfer of timing parameters from the timing design program to the field hardware.

Figure 4-3 PASSER II Movement Assignment Conventions



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 numbering convention of ***Design Standards, Index 17870*** on controllers that offer this alternative.

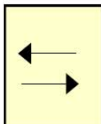
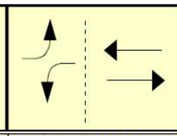
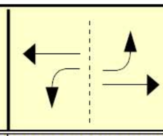
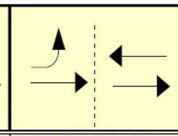
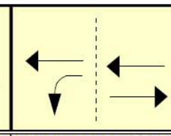
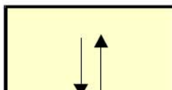
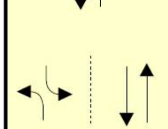
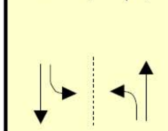
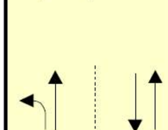
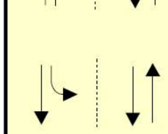
4.10.2 Identifying the Standard Signal Operating Plan

The standard operating plan (SOP) is normally used in place of the 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 5 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 one-way streets and "T" intersections (SOP 12 and 13) that are not represented on ***Table 4-2***. SOP's have also been developed for diamond interchanges (SOP 14, 15, 18, 19 and 20) and mid-block pedestrian signals (SOP 17). In addition, three preemption operating plans (POP's) are presented for application at railroad grade crossings.

Table 4-2 Standard 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

In some cases no SOP number has been assigned, as indicated by blank cells in ***Table 4-2***. 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 neither left turn in any opposing pair is protected (SOP 1,2,3,7 or 11), 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 and left turns, but they could influence the assignment of right turns. Right turns are normally accommodated by the same solid green display that controls the through movement. When right turns can be given adequate capacity by displaying a green right turn arrow concurrently with the shadowing left turn phase (e.g., northbound right and westbound left), a protected pedestrian phase may be created by prohibiting right turns on the through traffic phase.

4.11 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}***.

4.11.1 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,
4. Phasing plan selection and
5. Assignment of movements to timing functions.

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 and any unusual site characteristics and;
3. Constraints imposed by the system, especially the cycle length requirements of other intersections.

4.11.2 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.

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

> 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}.***

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 crossing time. The crossing time should be determined as L/Sp, where L is the crosswalk length and Sp is the assumed walking speed of pedestrians. The commonly accepted value of Sp is 4.0 ft./sec. This value should be used unless site-specific conditions suggest otherwise. For example, pedestrian walking speed depends on the proportion of elderly pedestrians (65 years of age and older) in the walking population. If a significant number of elderly pedestrians routinely cross at an intersection, a walking speed of 3.3 ft/s is recommended. The pedestrian crossing time will determine the length of the flashing don't walk pedestrian clearance interval for phases that have separate pedestrian control displays.

As suggested by the ***HCM***, the length of the pedestrian startup interval should be determined as:

3.2 (1 + Np /12)	where Np is the number of pedestrians per cycle, in the heaviest direction for crosswalks up to 12 feet in width and
3.2 (1 + Np /Wc)	where Wc is the crosswalk width, for crosswalks greater than 12 feet in width.

These computations should be subject to a minimum value of 5 seconds if no pedestrian control displays are provided for the phase. If pedestrian control displays are provided, the minimum value should be determined by the minimum length of the WALK interval.

4.11.3 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 time 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 4.11.5*** and ***4.11.6***. The common software products used for timing design and performance evaluation do not determine the intergreen parameters. These computations must be performed externally.

> The duration of the yellow change and all-red clearance intervals shall be predetermined ***{MUTCD}.***

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 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.

> 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}.***

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 their neighbors, 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.

4.11.4 Signal Timing Software Products

The 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, fuel consumption or progression quality as perceived by the motorist. Several such software products exist and are in common use. Those that are recognized by the FDOT include:

1. SOAP, which produces design timing and performance analysis for a single signalized intersections.
2. SIDRA, which is a popular international program developed in Australia. SIDRA also deals with single intersections, but extends the analysis to cover two-way stop and all-way stop control in addition to roundabouts. SIDRA is recognized as the most complex and credible of all the analytical intersection models.
3. SIGNAL 2000, which contains a signal design and analysis model compatible with the ***HCM*** analysis procedures. This program is a part of the TEAPAC package which offers additional functionality including the mapping of data into formats recognized by other models.

4. The Highway Capacity Software (HCS) that implements the ***HCM*** signalized intersection procedures explicitly. The HCS contains modules that analyze both signalized and unsignalized intersections.

The above products were designed principally for the analysis of individual intersection operations. Another group of products has been developed to deal with arterial and network traffic control systems. This group includes:

1. PASSER III, which extends the analysis of single intersections to include diamond interchanges.
2. PASSER II and IV, which perform timing design and performance analysis on arterial streets and network systems, respectively. Both programs attempt to maximize the quality of driver-perceived progression along a route.
3. TRANSYT-7F, which attempts to optimize the overall performance in terms of a user-specified disutility function that may consider delay, stops, fuel consumption, total operating cost and progression quality.
4. CORSIM, which is a microscopic simulation program that analyzes traffic control networks and freeways. CORSIM produces realistic animated graphics showing the movement of each vehicle as a part of the user interface. CORSIM has no internal optimization capability but, because of its microscopic simulation model, is able to accommodate unusual situations beyond the range of the deterministic models.
5. SYNCRO/SIMTRAFFIC is a privately developed set of software tools that includes the functionality of all of the products mentioned above.
6. ART-PLAN, which is a tool developed by the FDOT to perform a planning level analysis of an arterial route. ART-PLAN has no design or optimization capability but its input data requirements are minimal and it provides a useful alternative for determining whether or not a proposed design will provide an acceptable capacity and level of service.

This collection of design and analysis tools defines the methodology for signal timing design and analysis in Florida. It is not intended to preclude the use of other methods, especially those that reach the market in the future.

4.11.5 Yellow Change Interval Requirements

A yellow change interval is always required to terminate a green interval. In general, solid yellow indications follow solid green indications and yellow arrows follow green arrows. ***Figure 4-4*** shows the clearance table presented in ***Design Standards, Index 17870***. This table indicates the requirement for and the nature of the clearance display

> A steady yellow arrow shall be used to terminate a green arrow indication when the arrow controls a turn in an exclusive lane or when the arrow has been displayed concurrently with a circular green or red indication ***{MUTCD}.***

that must follow each display of a solid green or green arrow.

The yellow change interval must be long enough to avoid creating a “dilemma zone” which is defined as a condition under which the motorist can neither stop nor proceed through the intersection safely.

A widely used formula for determining the minimum length of yellow change interval to avoid the dilemma zone is:

$$Y = t + \nu/2\alpha$$

where:

Y = minimum yellow change interval length

t_r = reaction time, sec

ν= approach speed

α= deceleration rate

This formula is widely used and is known as the “ITE Formula.” 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***.

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

SIGNAL CLEARANCE TABLE

(Blank Indicates No Clearance Required)

To \ From: SIGNAL INDICATIONS	R	←R	G		←G	↑G	WALK	DONT WALK
R			Y		←Y	Y		
←R			Y		←Y	Y		
G					←Y			
←G								
↑G								
WALK								
DONT WALK							Flash DONT WALK	

(Row labels grouped under: SIGNAL INDICATIONS)

4.11.6 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:

$$AR=(w+l)/v$$

where:

w = width of intersection to be cleared

l = length of vehicle

v = approach speed

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.

4.11.7 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 5 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 5-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 4 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.

4.11.8 Interconnection Requirements

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 ½ mile on major routes should be coordinated ***{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.

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

4.12 Determination of the Detector Configuration

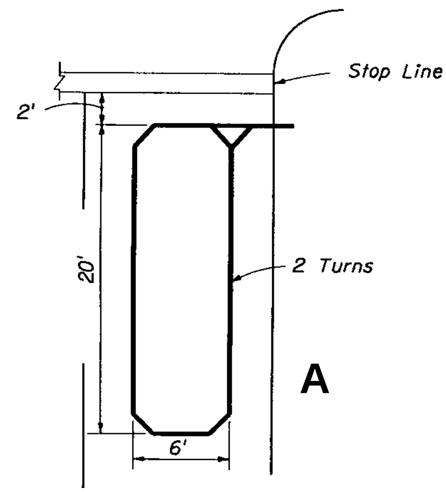
The detectors at a signalized intersection serve the singular purpose of informing the controller that a vehicle is (or is not) present on a particular approach to an intersection at any point in time. The controller uses this information to regulate the assignment of green time among competing movements 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.

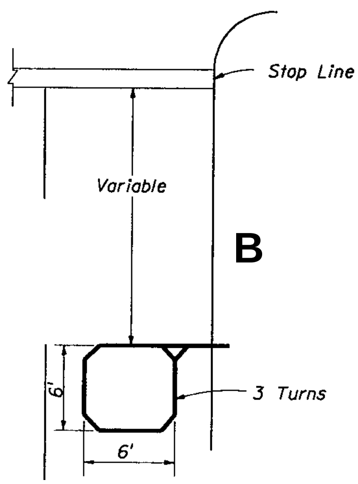
Vehicle detectors require FDOT traffic control device certification. There are several types of detectors on the market, each of which exploits a different principle of physics to detect the presence of a vehicle. The inductive loop detector has been the predominant detector type for more than 30 years 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. The detector types are summarized as follows:

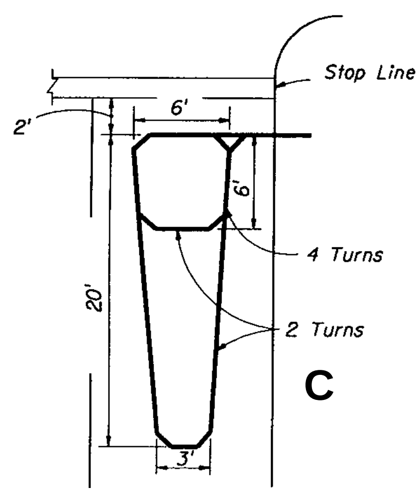
Type A Loops consist of a simple rectangular shape generally located at or close to the stop line. The standard dimensions are 6 feet by 20 feet. This is the simplest type of loop to install. Its main drawback is that the area of influence tends to extend well beyond its lateral boundaries, creating 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.

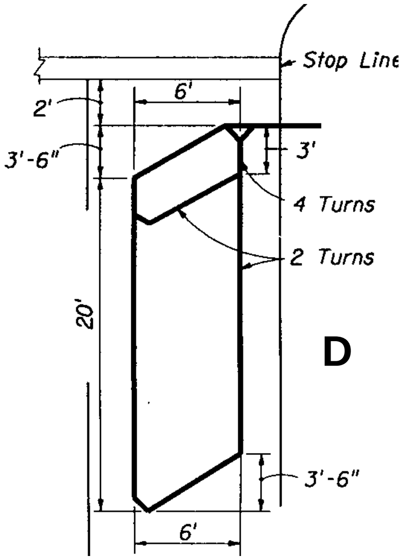




Type B Loops are very similar to Type A except that they are square and smaller in size. The standard dimensions are 6 feet by 6 feet. They do not generally cover sufficient area to detect all vehicles at the stop line because of the wide range of stopping positions. Type B loops are best suited to counting 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 to 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 to be predictable.

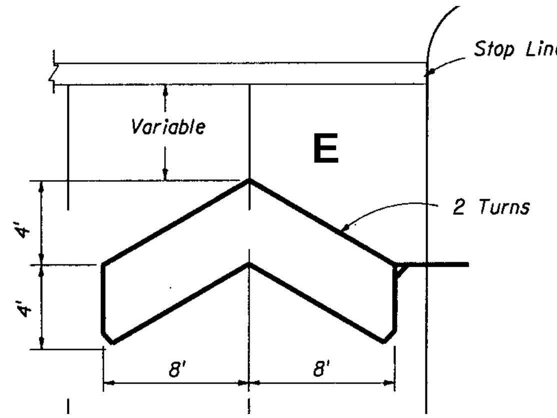
Type C Loops have the same general dimensions as Type A loops, except that they are trapezoidal in shape, instead of rectangular. They also have a supplementary loop consisting of a few turns at the downstream end to form a "booster" which increases sensitivity to bicycles. The trapezoidal shape results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

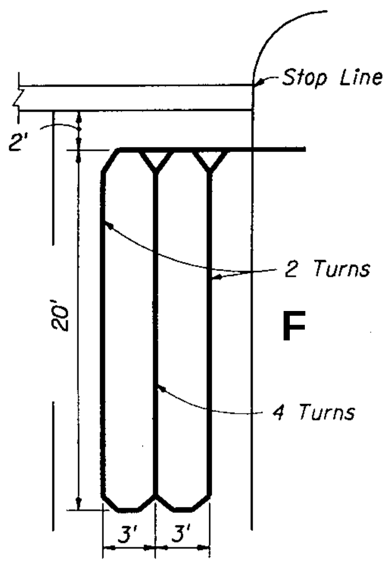




Type D Loops: have the same general dimensions as Type A and Type C loops, in a parallelogram shape. This shape implements the same principle of introducing an angular motion of the vehicle with respect to the loop wires to increase the sensitivity to bicycles and small vehicles.

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 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.

> 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 ***{Index 17781}.***

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

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.
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

Detectors based on other technologies have demonstrated their ability to perform acceptably, but generally at a higher cost. A variety of "above ground" devices based on infrared, microwave and ultrasonic transducers is readily available. The choice of these devices for design purposes is normally limited to situations that preclude the installation of a loop in the roadway (e.g., unpaved roads, bridge decks, etc.). An emerging problem for inductive loop technology is the detection of "space age" bicycles with little or no metallic content.

Video image detection (VID) offers an alternative vehicle detection technology for intersections. One video camera can cover several lanes of traffic and studies have shown that the detection accuracy is comparable to loops. As the cost of this technology falls, VID may capture an increasing share of the detection market in the future.

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.

1. Pedestrian detector mounting height should be approx. 3.5 feet
2. Explanatory signs required
3. Applicable crosswalk should be clearly indicated
4. Confirmation signal (e.g., pilot light) may be provided ***{MUTCD}***.

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***.

Figure 4-5 Typical Pedestrian Push Button Detectors

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 a 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 strategically 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 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 traveled roadway 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 decisions with respect to vehicular signal displays include:

1. Horizontal or vertical mounting,
2. Size of lenses (8 or 12 inches),
3. Indication for each lens (solid or arrow) and
4. Position of each signal head.

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

1. A minimum of two signal faces shall be provided for the major movement on the approach, even if the major movement is a turning movement. Except where the width of an intersecting roadway or other conditions make it physically impractical, at least one and preferably both of the signal faces shall be located to conform to the positioning requirements shown in ***Figure 4-7***. Three aspects of positioning are illustrated on this figure:

    A. A diagram shows the maximum horizontal position with respect to the center of the approach at the stop line;

B. A table shows the minimum sight distance as a function of the approach speed and

C. A graph shows the minimum mounting height to the top of the signal head as a function of the distance from the stop line. Note that previous editions of the ***MUTCD*** specified the minimum mounting height to the bottom of the signal head.

2. Twelve-inch signal lenses are normally installed at signalized intersection on the State Highway System. The ***MUTCD*** requires that they shall be used:

   A. Where road users view both traffic control and lane-use control signal heads simultaneously;

   B. If the nearest signal face is between 120 feet and 150 feet beyond the stop line, unless a supplemental near-side signal face is provided;

   C. For all signal faces located more than 150 feet from the stop line;

   D. For approaches to all signalized locations for which the minimum sight distance in ***Figure 4-7*** cannot be met;

   E. For arrow signal indications and

   F. In a circular red indication when 12 inch lenses have been used for green or yellow indications in the same signal face.

3. Twelve-inch signal lenses should be used:

   A. On approaches with 85th-percentile speeds exceeding 40 mph;

   B. On approaches where a traffic control signal might be unexpected;

   C. On all approaches without curbs and gutters where only post-mounted signal heads are used and

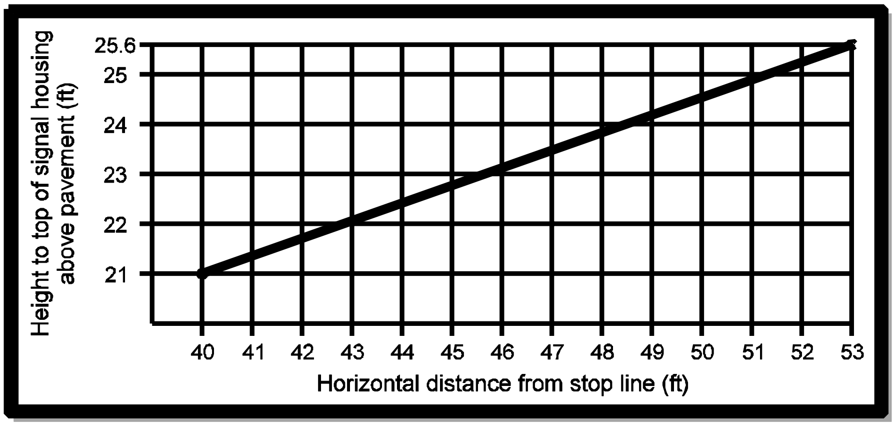
   D. At locations where there is a significant percentage of elderly drivers.

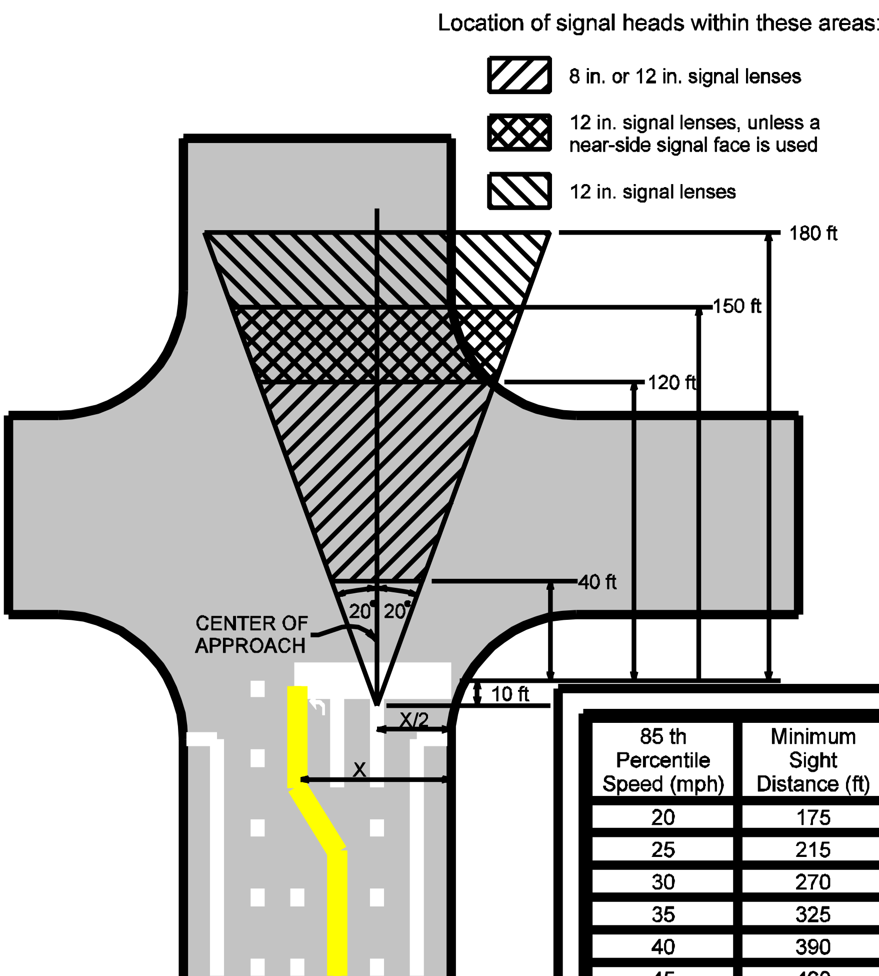
Each vehicular signal face shall have three, four or five signal sections. Single-section signal faces are only permitted for continuously illuminated GREEN ARROW indications. The proper relative positions of signal lenses within the signal face are shown in ***Figure 4-6***.

Figure 4-6 Order of Signal Lenses in Horizontally and Vertically Mounted Heads

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 4-7 MUTCD Requirements for Location, Maximum Mounting Height and Minimum Sight Distance of Signal Displays.





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

If the minimum sight distance in ***Figure 4-7*** cannot be met, a sign shall be installed to warn approaching traffic of the traffic control signal.

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 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.

> 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}.***

Nearside signal faces should be located as near 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.

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

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.

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 dull 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).

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

1. If a traffic control signal is justified on the basis of pedestrian volume or a school crossing;
2. If an exclusive pedestrian phase is provided;
3. At an established school crossing at any signalized location and
4. When multi-phase signal indications would tend to confuse pedestrians guided only by vehicular signal indications.

Pedestrian signal heads should be used:

1. If it is necessary to assist pedestrians in making a reasonably safe crossing 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.

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

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 alongside 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.

> 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}.***

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, then an examination should ensue that considers 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, 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 who 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 requirements for three types of accessible pedestrian signals:

1. Audible Tone Signals

   When accessible pedestrian signals have an audible tone(s), they shall have a tone for the walk interval. The audible tone(s) shall be audible from the beginning of the associated crosswalk. If the tone for the walk interval is similar to the pushbutton locator tone, the walk interval tone shall have a faster repetition rate than the associated pushbutton locator tone.

   When choosing audible tones, possible extraneous sources of sounds (such as wind, rain, vehicle back-up warnings or birds) shall be considered in order to eliminate potential confusion to pedestrians who have visual disabilities. Audible pedestrian tones should be carefully selected to avoid misleading pedestrians who have visual disabilities when the following conditions exist:

   A. Where there is an island that allows unsignalized right turns across a crosswalk between the island and the sidewalk;

   B. 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 or

   C. 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.

   A pushbutton locator tone is a repeating sound that informs approaching pedestrians that they are required to push a button to actuate pedestrian timing and that enables visually impaired pedestrians to locate the pushbutton. The accessible walk signal tone should be no louder than the locator tone, except when there is optional activation to provide a louder signal tone for a single pedestrian phase.

2. Verbal Message Signals

   When verbal messages are used to communicate the pedestrian interval, they shall provide a clear message that the walk interval is in effect, as well as to which crossing it applies. The verbal message that is provided at regular intervals throughout the timing of the walk interval shall be the term "walk sign," which may be followed by the name of the street to be crossed. A verbal message is not required at times when the walk interval is not timing, but, if provided:

   A. It shall be the term “wait,” and

   B. It need not be repeated for the entire time that the walk interval is not timing.

3. Vibrotactile Signals

   A vibrotactile pedestrian device communicates information about pedestrian timing through a vibrating surface by touch. Where used, it shall indicate that the walk interval is in effect and for which direction it applies, through the use of a vibrating directional arrow or some other means. Vibrotactile pedestrian devices should be located next to and on the same pole as the pedestrian pushbutton, if any, and adjacent to the intended 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.

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.

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 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.

4.15 Equipment Required to Support the Signal Displays

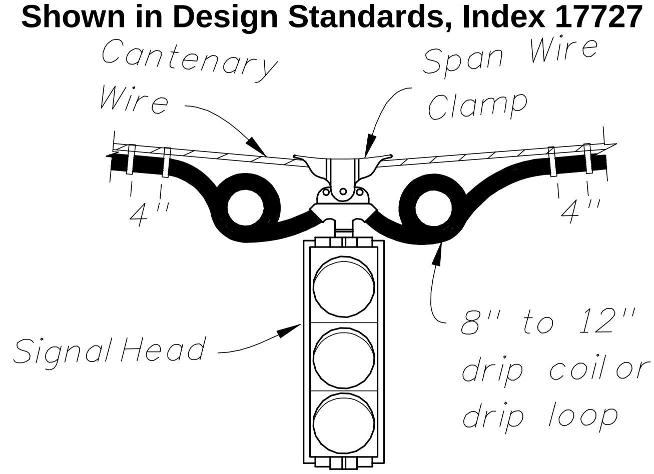
Signal displays may be mounted either on mast arms or span wires. It is a requirement in Florida that all signal displays within 10 miles of the coastline be mounted on mast arms because of the susceptibility of span wire mounting to high winds. Flashing beacon displays are exempt from this requirement. Typical horizontal (mast arm) and vertical (span-wire) installations are shown in ***Figure 4-8***.

Figure 4-8 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 concrete and steel 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 4-9***.

Figure 4-9 Typical Span-Mounted Signal Head as Shown in Design Standards, Index 17727



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 17740 and 17745*** provide instructions, examples, component data and installation requirements for mast arms.
6. ***Index 17746*** provides design details, elevation details, camber details and other instructions and notes concerning monotube signal structures.
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.

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

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.

> All new signals installed on the State Highway System along designated coastal evacuation routes, along the Strategic Intermodal System routes and corridors within ten miles of the coastline shall be supported by mast arms with signal heads rigidly attached to the mast arm ***{PPM}.***

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

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

1. Reference should be made to ***PPM Volume I, Chapter 2, Section 2.11*** and to the Americans with Disabilities Act.

2. Signal supports shall not be located in medians. On rural and urban flush shoulders they should be located outside the clear zone. On urban curb or curb and gutter 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 such that an unobstructed sidewalk width of 4 feet or more (not including the width of the curb) is provided.

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

3. No part of a concrete base for a signal support should extend more than 4 inches above the ground level at any point. This limitation does not apply to the concrete base for a rigid support.

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

Figures

Tables

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Chapter 5

Signs and Markings

5.1 General

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 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 of Standard Highway Signs*** provides information on sizes, 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***, (Washington, D.C., 1999), 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***. Reflective 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 Sections 700-4 and 700-5 of 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 II and III 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.

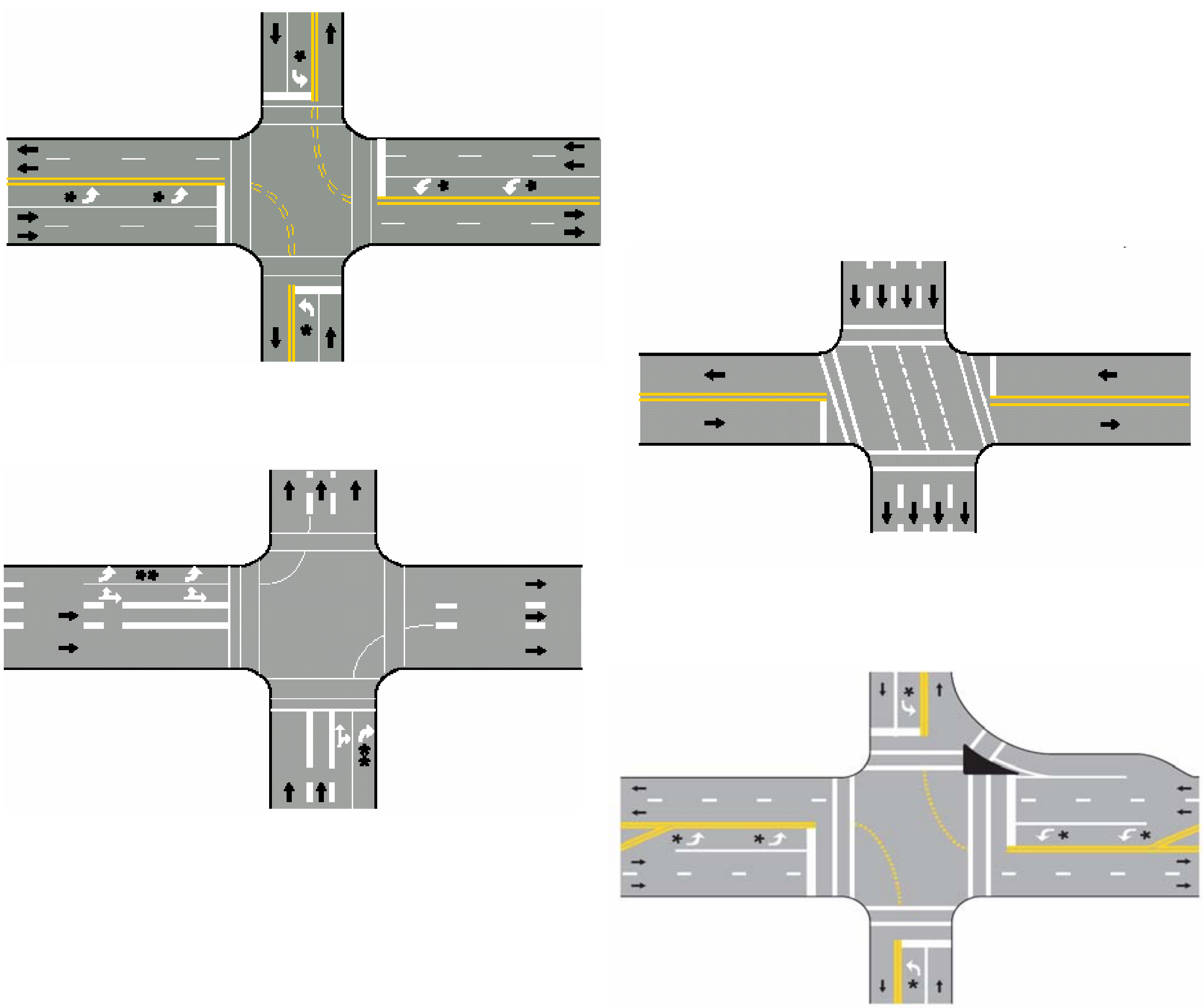
The effectiveness of pavement markings can be enhanced by the addition of audible and tactile features such as bars, differential surface profiles, raised pavement markers or other devices that may alert the road user that a boundary of the roadway is being traversed.

The typical markings prescribed by the ***MUTCD*** for intersections are shown in ***Figure 5-1***. The criteria for line widths and spacings are presented in ***Table 5-1***.

Table 5-1 Pavement Marking Dimensions

Widths	6 in.	Normal longitudinal lane markings
	8 in.	Painted channelizing lines
	12 in.	Normal crosswalk lines (longitudinal to crosswalk)
		Special emphasis crosswalk lines (transverse to crosswalk)
	18 in.	Gore area transverse (diagonal and chevron pattern)
	24 in.	Stop line
Spacing	4 in.	Between double line markings
	24 in.	Between special emphasis crosswalk lines (transverse)
Skip pattern	10 ft./30 ft.	Normal longitudinal skip line
(line/skip)	6 ft./10 ft.	lane line extensions through crossover area
	2 ft./4 ft.	lane line extensions within intersections

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



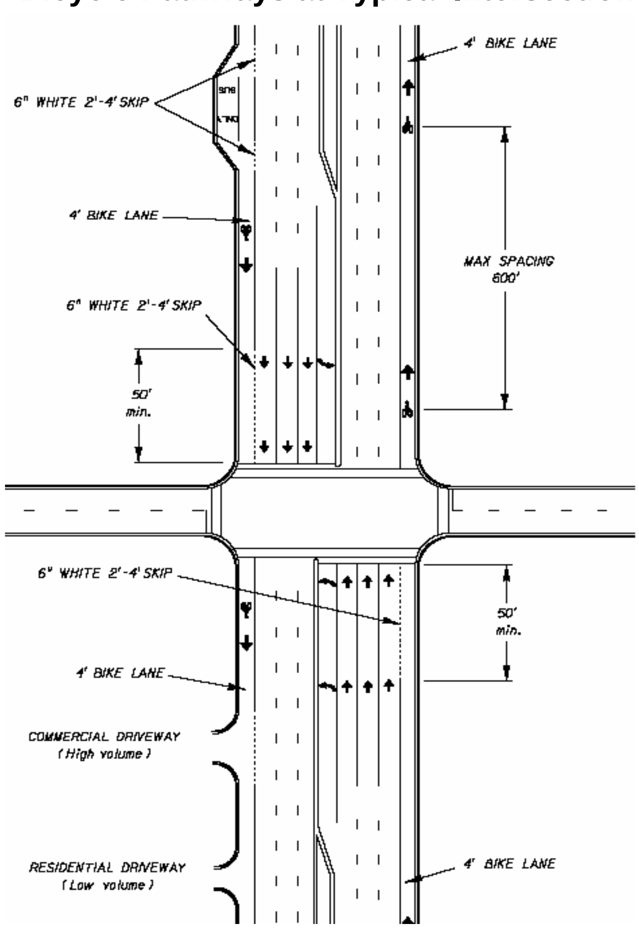
5.4.3 Special Marking Requirements for Bicycle Lanes

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 or at multi lane roundabouts. 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 foot 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. (See ***Figure 5-2***)

Figure 5-2 Bicycle Pathways at Typical Intersections)



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 orientation, guidance and restrictions at each intersection and in advance of the intersection. Orientation is provided as street signs. Markings provide guidance cues and restrictions or prohibitions in the form of regulatory signs (e.g., “Do Not Enter”).

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. Larger letter heights should be used for street name signs mounted overhead. The ***TEM*** extends the ***MUTCD*** criteria. Specifically, the ***TEM*** requires that lettering on street name signs be UPPER CASE and should be at least 8 inches high. 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. 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.

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

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

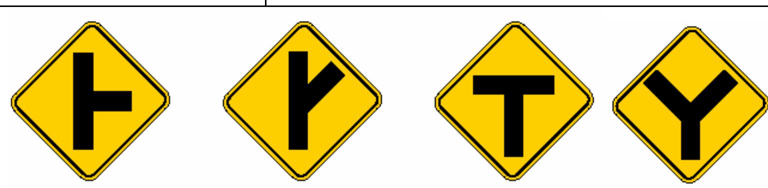
When sight distance is limited or transitory events are likely to be encountered, warning signs should be erected prior to affected intersections. These signs should be placed along the street at a distance sufficient for the driver to perceive, identify, decide and perform necessary maneuvers. The time 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 5-2*** summarizes the more critical advance signs needed at intersections.

> 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}.***

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 5-3*** 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 5-2 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 Yield sign not visible at sufficient distance that allows vehicle to stop at Yield Sign.
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.
Side Road, T Symbol or Y Symbol (W2-2, W2-3, W2-4 and W2-5		Warns of traffic from directions indicated in symbol

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.

Table 5-3 Guide for Estimating Advanced 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[4]							
		0[3]	10	20	30	40	50	60	70
20 mph	225 ft	N/A[5]	N/A[5]	—	—	—	—	—	—
25 mph	325 ft	N/A[5]	N/A[5]	N/A[5]	—	—	—	—	—
30 mph	450 ft	N/A[5]	N/A[5]	N/A[5]	—	—	—	—	—
35 mph	550 ft	N/A[5]	N/A[5]	N/A[5]	N/A[5]	—	—	—	—
40 mph	650 ft	125 ft	N/A[5]	N/A[5]	N/A[5]	—	—	—	—
45 mph	750 ft	175 ft	125 ft	N/A[5]	N/A[5]	N/A[5]	—	—	—
50 mph	850 ft	250 ft	200 ft	150 ft	100 ft	N/A[5]	—	—	—
55 mph	950 ft	325 ft	275 ft	225 ft	175 ft	100 ft	N/A[5]	—	—
60 mph	1100 ft	400 ft	350 ft	300 ft	250 ft	175 ft	N/A[5]	—	—
65 mph	1200 ft	475 ft	425 ft	400 ft	350 ft	275 ft	175 ft	N/A[5]	—
70 mph	1250 ft	550 ft	525 ft	500 ft	425 ft	350 ft	250 ft	150 ft	—
75 mph	1350 ft	650 ft	625 ft	600 ft	525 ft	450 ft	350 ft	250 ft	100 ft

Notes:

[1] The distances are adjusted for a sign legibility distance of 175 ft for Condition A. The distances for Condition B have been adjusted for a sign legibility distance of 250 ft, which is appropriate for an alignment warning symbol 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 PIEV time of 14.0 to 14.5 seconds for vehicle maneuvers (2001 AASHTO Policy, Exhibit 3-3, Decision Sight Distance, Avoidance Maneuver E) minus the legibility distance of 175 ft 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 2001 AASHTO Policy, Stopping Sight Distance, Exhibit 3-1, providing a PIEV time of 2.5 seconds, a deceleration rate of 11.2 ft/second^2, minus the sign legibility distance of 175 ft.

[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 PIEV time, a vehicle deceleration rate of 10 ft/second^2, minus the sign legibility distance of 250 ft.

[5] No suggested distances are provided for these speeds, as the placement location is dependent on site conditions and other signing to provide an adequate advance warning for the driver.

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 used, 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 ***{MUTCD}.***

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. Note that the ***MUTCD*** prescribes a new type of 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.

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.

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}.***

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.

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.

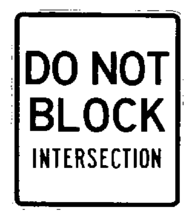
A NO TURN ON RED sign may be considered whenever an engineering study determines that one or more of the following conditions exist:

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. They are not common on the SHS. 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

Advance non-vehicular 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.

> Non-vehicular signs should be used only at locations where the crossing activity is unexpected or at locations not readily apparent ***{MUTCD}.***

If the location of the crosswalk is not apparent and therefore requires emphasis, a second advance non-vehicular 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 and that a driver is merely being directed progressively to 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 Bays

Where traffic volumes, roadway design or reduced visibility conditions warrant the use of right and/or left turn bays, 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.

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 bay. 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 allowed turning movement.

> 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}.***

Figure 5-3 shows the markings for a single-lane right turn bay. ***Figure 5-4*** illustrates the proper treatment of markings for left turn bays. Please refer to ***Chapter 3, Section 3.11*** for a discussion of the geometric design aspects of turn bays.

Figure 5-3 Markings and Channelization for Single-Lane Left Turn Bays

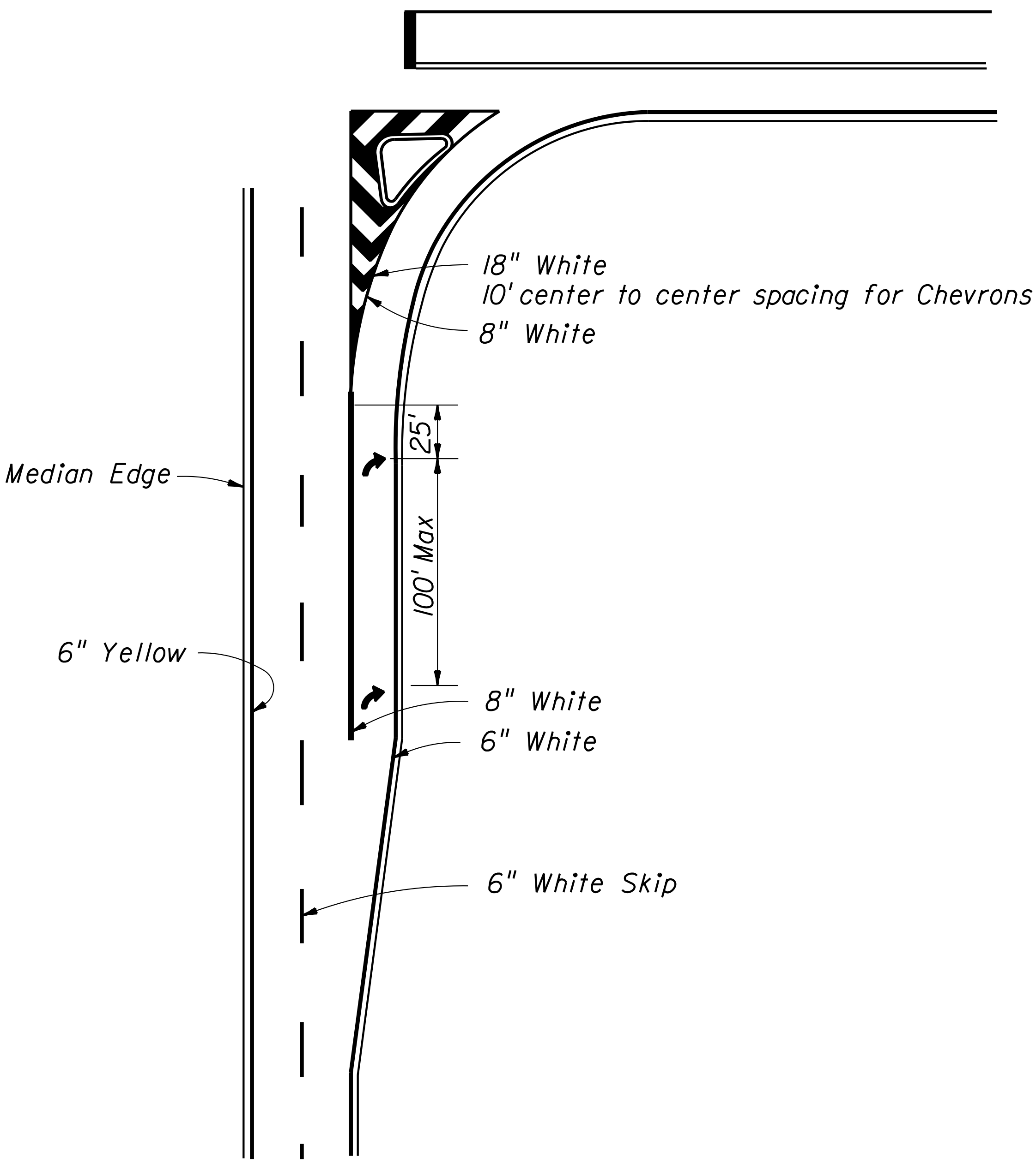
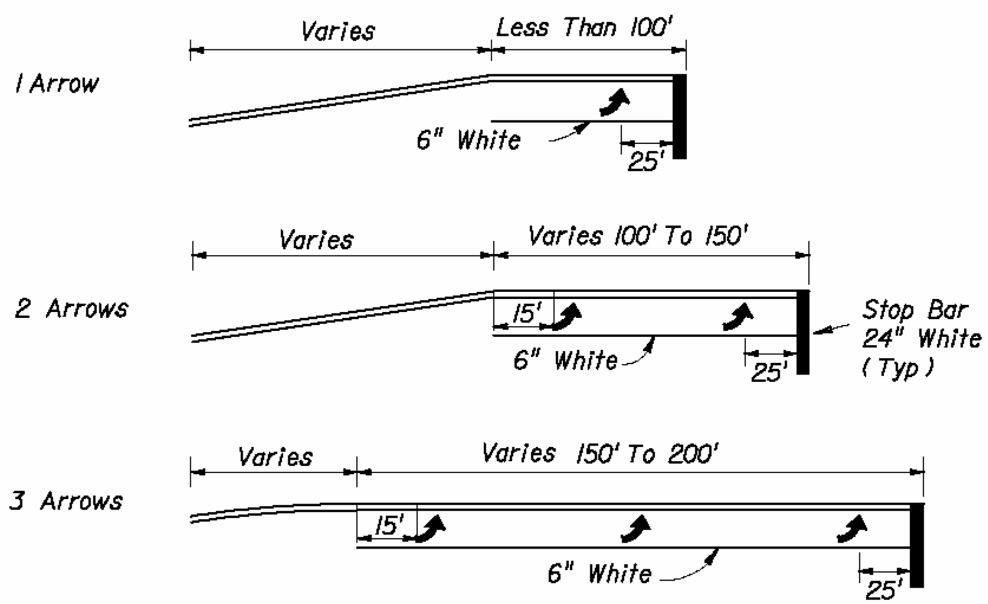


Figure 5-4 Marking of Lanes with Left Turn Bays



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 drivers 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 5-5 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. Details of the markings at the intersection are provided in ***Design Standards, Index 17346***.

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.

Table 5-4 summarizes the various signs that are commonly used in school zones. A plaque indicating "AHEAD" or, optionally, "XXX FT" supplements the standard "School Advance Warning Sign".

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 the S1-1 signs on FDOT projects in accordance with ***FDOT Specifications, Section 700-2.5***.

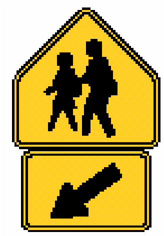


Figure 5-5 Typical Signs and Markings for a School Crosswalk at a Signalized Intersection

Approach Speed (MPH)	Distance A (FT)
25 or less	200
26 To 35	250
36 To 45	300
46 to 55	325

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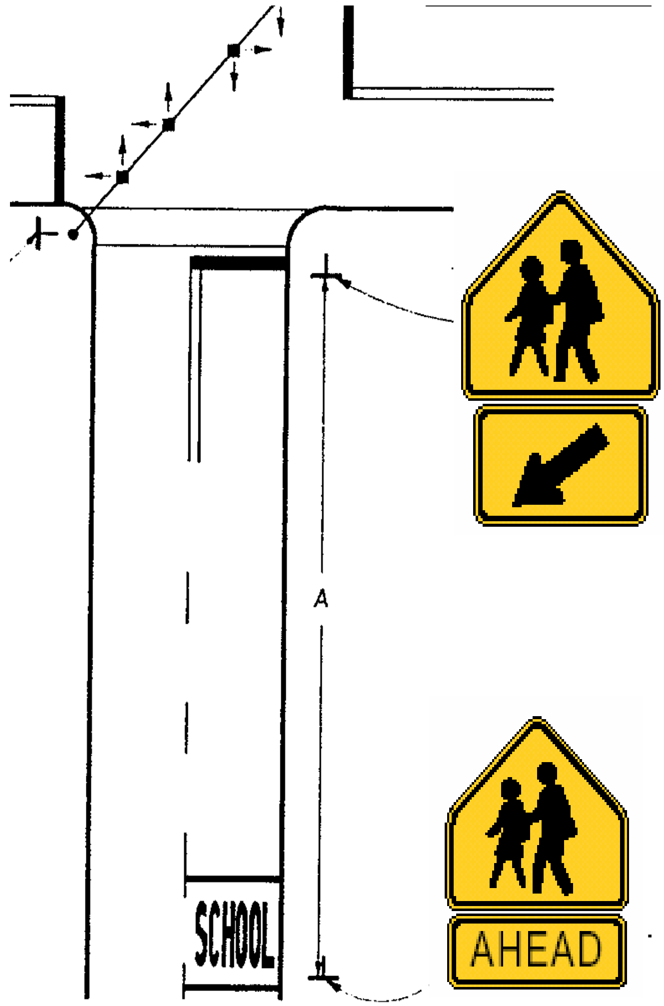


Notes:

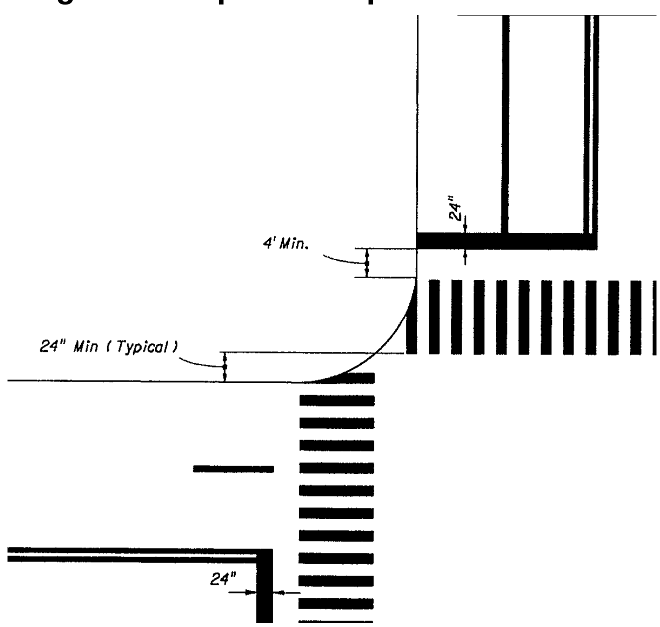
1. The school crosswalk warning assembly shall not be installed on stop-controlled approaches.
2. The supplemental plaque for the advance warning sign may read “XXX FT” or “AHEAD.”
3. “No Right Turn On Red” signs may be erected as deemed necessary by the local traffic engineers.

Table 5-4 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. m	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 picked up and discharged and where visibility 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

Figure 5-6 Special Emphasis Crosswalk



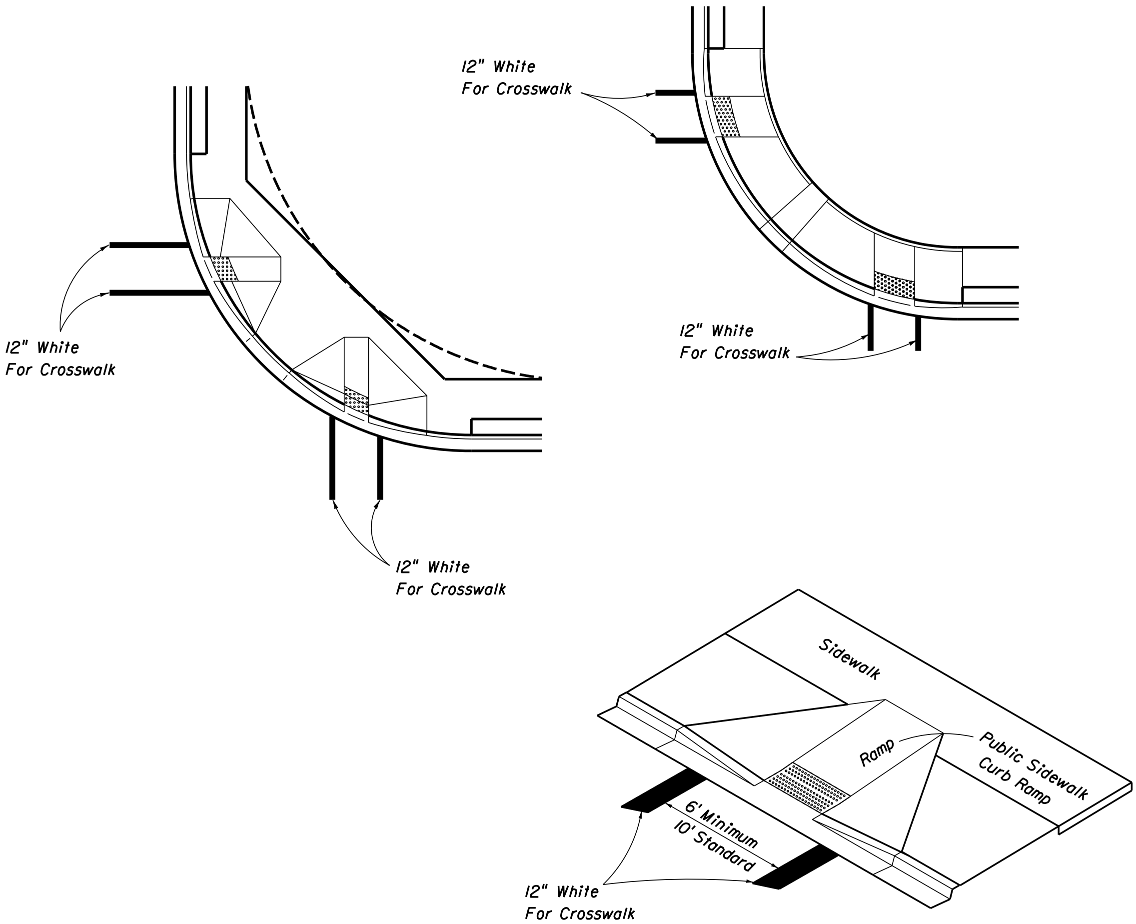
Crosswalk lines are solid white lines marking both edges of the crosswalk. The ***MUTCD*** requires a minimum width of 6 inches. The standard width for the SHS is 12 inches. They should be spaced 6 feet apart as illustrated in ***Design Standards, Index 17346***. Where no advanced stop line is provided or where vehicular speeds exceed 30 mph or where crosswalks are unexpected, the width of the crosswalk line should be 24 inches wide. Crosswalk lines on both sides of the crosswalk should extend across the full width of pavement to discourage diagonal walking between crosswalks. In school locations where traffic volumes are high and there are nearby businesses and business driveways, pedestrian crossings may need to be made even more conspicuous. These configurations are referred to as "special emphasis crosswalks" and are illustrated in ***Figure 5-6***.

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.

For added visibility, the area of the crosswalk may be marked with white diagonal lines at a 45-degree angle or with white longitudinal lines at a 90-degree angle to the line of the crosswalk. These lines should be 12 inches wide and spaced 24 inches apart, as illustrated in ***Design Standards, Index 17346***. When diagonal or longitudinal lines are used to mark a crosswalk, the transverse crosswalk lines may be omitted. Care should be taken to insure that crosswalks with diagonal or longitudinal lines used at some locations do not detract from other crosswalks where special emphasis markings are not used.

Typical crosswalk markings for public sidewalk curb ramps are provided in ***Design Standards, Index 17346*** and illustrated in ***Figure 5-7***. Standard details for special emphasis crosswalks are also provided in ***Design Standards, Index 17346***.

Figure 5-7 Typical Crosswalk Markings for Public Sidewalk Curb Ramps

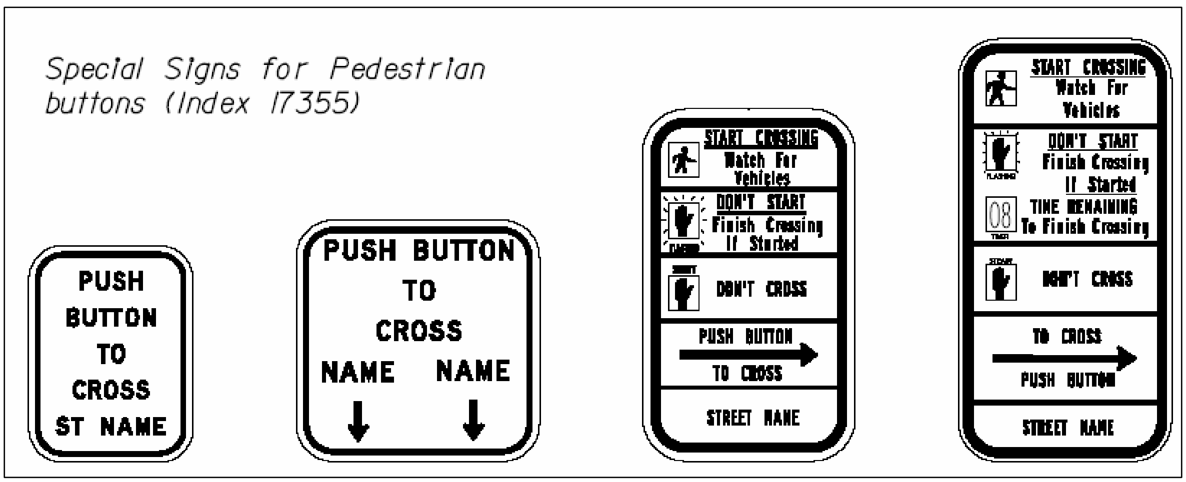


5.9.3 Pedestrian Button Signs

Pedestrian push buttons are normally installed to ensure that the full sequence of pedestrian indications is displayed upon demand. These push buttons should be conveniently 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 ***Design Standards, Index 17355***.

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

Figure 5-8 Special Signs for Pedestrian Buttons



5.9.4 Special Populations

The special needs of those with particular limitations must be considered. This includes persons who have visual, auditory and ambulatory limitations. It also includes our senior citizens who may have a combination of limitations. The most important considerations for special populations are summarized as follows:

1. Special Emphasis Crosswalks: For older populations, special emphasis is needed on design features of crosswalks. The same treatments used for special emphasis crosswalks in school zones are merited in areas where there are substantial numbers of older persons (See ***Figure 5-6***).

2. Use of Tactile Surfaces: Work performed for FHWA in the early 1980s indicated the guidance benefits of tactile surfaces. This work has not been widely used for the design of streets, but has been used for the design of subway terminal platforms. For areas where there are concentrations of people with visual limitations, tactile surface materials may be able to help with guidance to and into

crosswalks. Special "corduroy" tiles have been shown to be highly detectable when used in association with brushed concrete and coarse aggregate concrete.

3. Use of Transverse Thermoplastic in Crosswalks: There have been antidotal reports of mobility-impaired and wheelchair occupants having difficulty with tripping and falling over peeling thermal plastic materials. Properly install materials should not peel and thus would not cause such problems. In any case, there is not sufficient documentation of problems with these materials to merit restrictions on their use.
4. Crosswalks Across Slip Lanes: Slip lanes provide a clear advantage for the movement of vehicular traffic but pose difficulties for pedestrians, especially blind pedestrians. At locations where they are merited for the movement of vehicles, they should be used in conjunction with audible signals.
5. Crosswalk Termination at a Point of Safe Refuge: Crosswalks on wide streets with raised medians or painted refuge areas should be terminated at the edge of these areas. They should not be painted through the refuge area.

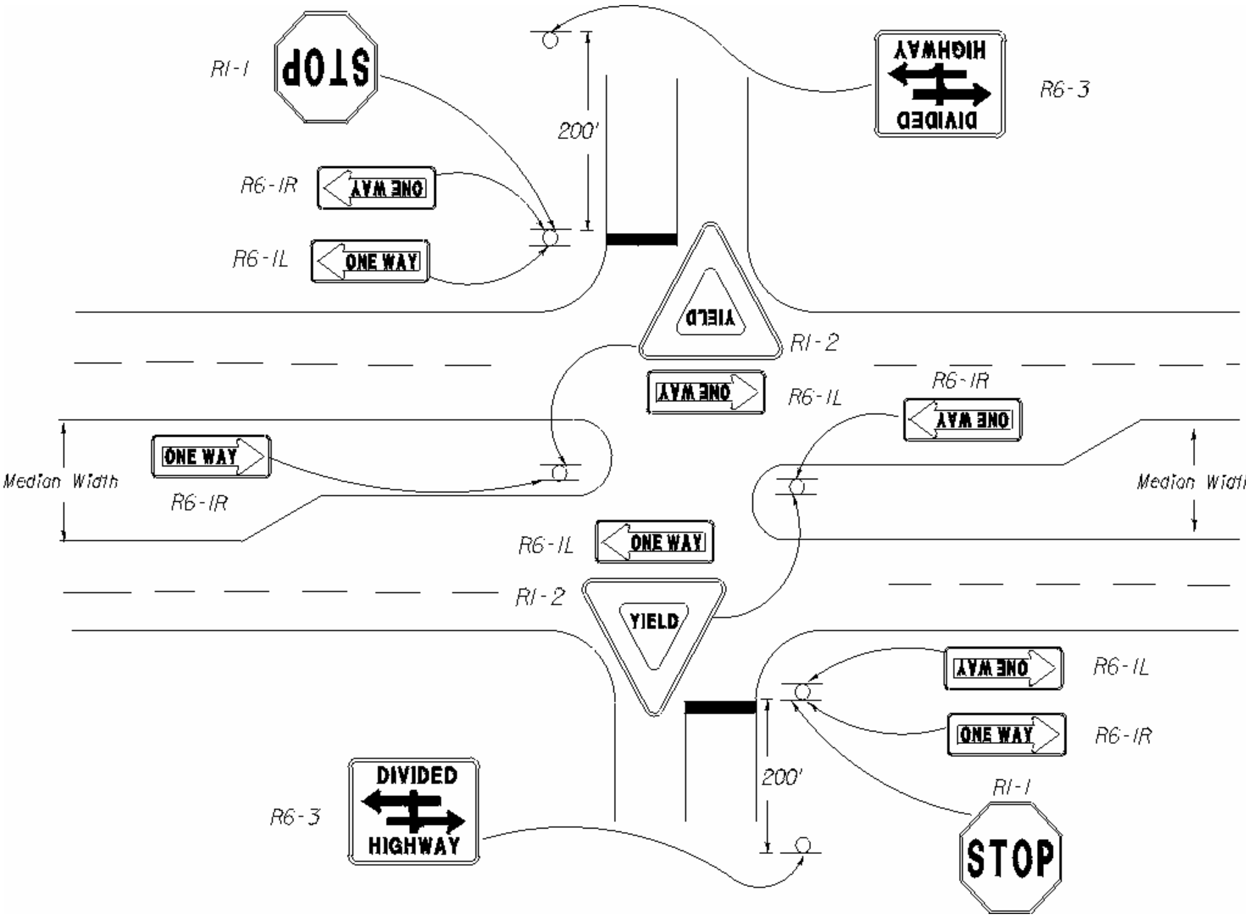
5.10 Signs and Markings for Medians and Divided Highways

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 5-9***. A distinction is made between divided highways with medians less than 30 feet in width and those with medians wider than 30 feet. ***Figure 5-9*** shows the case of an intersection with a wide median as presented in Design Standards, ***Index 17346***. 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.

5.11 Roundabout Signs and Markings

Roundabouts have been used overseas for many years. They have been redesigned for use in the United States as an alternative to signalized intersections where traffic volumes are moderate and expected to remain at moderate levels. The planning and design of roundabouts is covered in more detail in the ***Florida Roundabout Guide***.

Figure 5-9 Typical Signs and Markings for an Intersection on a Divided Highway with Median Width Greater than 30 Feet



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***).

Figure 5-10 Advance Warning Sign for Roundabouts (W2-6)

Note the ***MUTCD*** has prescribed a standard symbol for advance warning signs at roundabouts. The use of this format is encouraged.

5.11.2 Entry Treatment

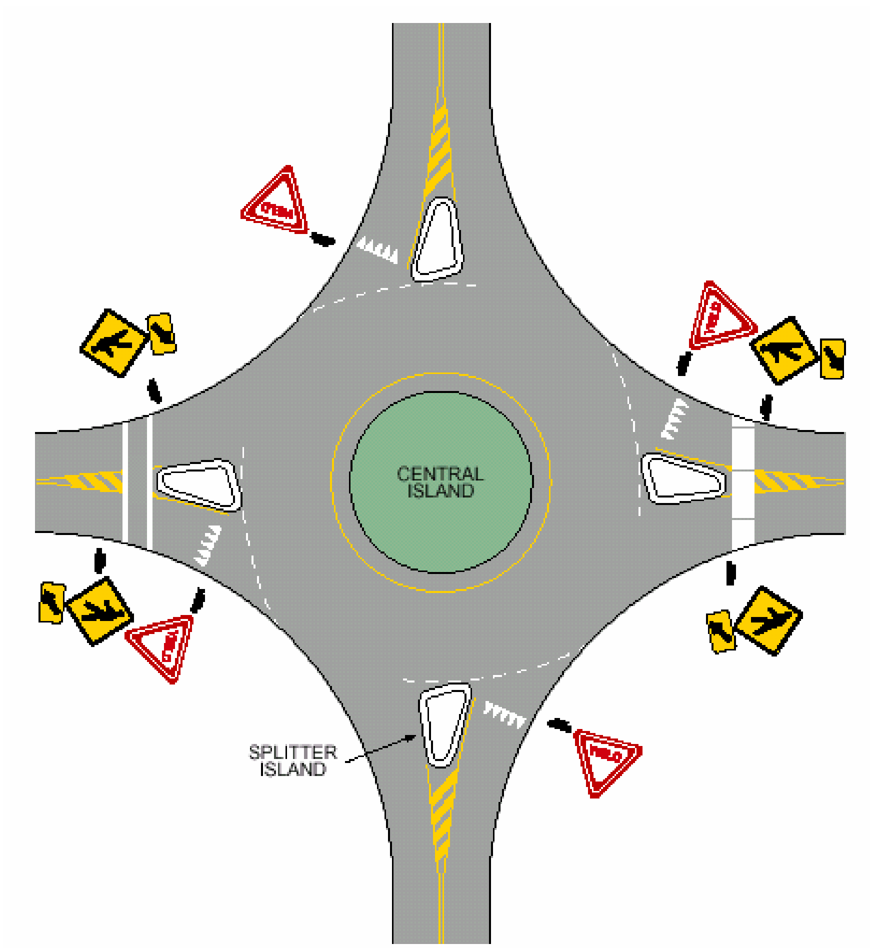
Figure 5-11 illustrates the entry treatment prescribed for roundabouts by the ***MUTCD***. Note that the new form of yield marking (i.e., triangles) is indicated on this figure. A YIELD sign is required at the entrance to the roundabout. Yield lines are required at the entry point of each approach to a roundabout. They should consist of 8-inch markings, 20 inches long with 20-inch gaps. There should be no painted lines across the exits from a roundabout. A YIELD legend marking is optional.

5.11.3 Circulatory Roadway

Standard signing inside the inscribed circle includes a standard ONE-WAY sign across from each approach. Directional chevron signs beneath the ONE-WAY signs are optional.

Marker signs for streets should be provided and, on marked state routes, trailblazer signs should be provided to guide the motorist through the roundabout.

Figure 5-11 Roundabout Signing and Marking Requirements (MUTCD)



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 generally needed. Approach roadways that have bicycle lanes should end and allow merging within the last 60 to 100 feet of the approach.

The provisions for pedestrians do not change the geometric design requirements from treatments required for other intersections. However, large roundabouts can result in greater walking distances, but special crossing facilities are not necessary. Well designed splitter islands to store 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 5-11***.

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

Figures

Tables

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Chapter 6

Objects and Amenities

6.0 General

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:

1. ***AASHTO Roadway Lighting Design Guide***;
2. ***NCHRP 152 "Warrants for Highway Lighting"***;
3. ***FDOT Highway Landscape Guide***;
4. ***Utility Accommodations Manual (UAM)***;
5. ***A Policy on Geometric Design of Highways and Streets (AASHTO Green Book)***;
6. ***Florida Manual of Uniform Minimum Standards for Design, Construction and Maintenance for Streets and Highways (Florida Greenbook)***;
7. ***FDOT Plans Preparation Manual***;
8. ***Transit Capacity and Quality of Service Manual (TRB Publication)***
9. ***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, decorative paving, fences and lighting (excluding public utility street and area lighting). Plans for landscape projects must be prepared by a landscape architect licensed by the State of Florida. The central objective of highway beautification is to create safe, low maintenance, attractive landscaping within the rights of way of the State's transportation corridors.

In addition to beautifying the highway, selective or well designed landscaping can help make roads safer for pedestrians, make roads more pedestrian friendly, reduce maintenance costs, reduce stormwater runoff, increase habitat connectivity and promote community values.

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. 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***.

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 17503 shows concrete foundation and pull box wiring details for metal poles.

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 the sign legend 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.

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.

Poles shall not be located in the median except in conjunction with barriers that are justified for other reasons.

The installed lighting system should have a pleasant daytime appearance and reflect aesthetic considerations.

6.3.2 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, which 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 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 FDOT 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.

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 crossings;
3. Maintenance of the utility, vegetation, etc.;
4. Maintenance of traffic;
5. Location criteria;
6. Utility surveys;
7. Special requirements and exceptions and
8. Definitions and acronyms.

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, arterials must also serve the adjacent land development and this function may result in the use of parking on the street.

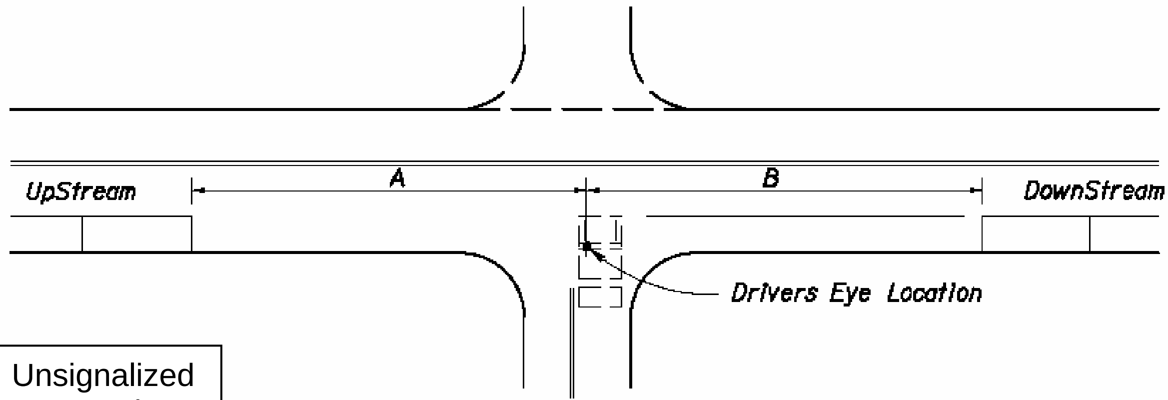
6.5.1 Relationship to Intersections

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.

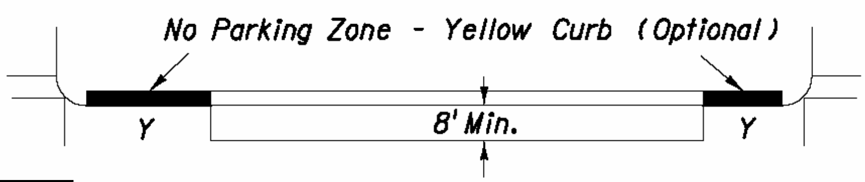
Where bicycle routes have been designated where parking is present, the bike lane should be placed between the parking lane and the travel lane and have a minimum width of 5 feet.

Figure 6-1 Minimum Parking Clearances for Signalized and Unsignalized Intersections as Specified in Design Standards, Index 17346.



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)

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.

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. Thus, the careful planning, design and traffic control measures of this and other forms of through traffic interference is crucial in the development of an urban highway design or improvement program.

On roadways where transit operates the lanes should be wide enough for the bus to stop safely. Through traffic may have to go into an opposing lane to pass the bus or when the bus turns the corner the front end may overhang into opposing lanes. The latter should be a consideration when placing stop lines.

6.6.2 Location of Waiting Facilities

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.

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.

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 be the same as the roadway.
6. Maximum slope: (2%) perpendicular to the roadway

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.

Bus stops on cross streets should be located and arranged so that transferring riders are not required to cross the artery, regardless of the direction they wish to travel.

Another form of bus stops is found on arterial streets with frontage roads where buses leave and return to the artery by special openings in the outer separation in advance of and beyond the intersection. Thus, buses stop in positions well removed from the through lanes. Right-turning traffic to and from the arterial street may also use 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 out should be considered, for pedestrians and persons in wheelchairs. If roadways are to be widened, transit shelters should be set back instead of eliminated.

Table 6-1 Comparative Analysis of Bus Stop Locations

(Source: TCRP Report 19)

	Advantages	Disadvantages
Far-Side Stop	1. Minimizes conflicts between right turning vehicles and buses 2. Provides additional right turn capacity by making curb lane available for traffic 3. Minimizes sight distance problems on approaches to intersection 4. Encourages pedestrians to cross behind the bus 5. Creates shorter deceleration distances for buses since the bus can use the intersection to decelerate 6. Results in bus drivers being able to take advantage of the gaps in traffic flow that are created at signalized intersections	1. May result in the intersections being blocked during peak periods by stopping buses 2. May obscure sight distance for crossing vehicles 3. May increase sight distance problems for crossing pedestrians 4. Can cause a bus to stop far side after stopping for a red light, which interferes with both bus operations and all other traffic 5. May increase number of rear-end accidents since drivers do not expect buses to stop again after stopping at a red light 6. Could result in traffic queued into intersection when a bus is stopped in travel lane
Near-Side Stop	1. Minimizes interferences when traffic is heavy on the far side of the intersection 2. Allows passengers to access buses closest to crosswalk 3. Results in the width of the intersection being available for the driver to pull away from curb 4. Eliminates the potential of double stopping 5. Allows passengers to board and alight while the bus is stopped at a red light 6. Provides driver with the opportunity to look for oncoming traffic, including other buses with potential passengers	1. Increases conflicts with right-turning vehicles 2. May result in stopped buses obscuring curbside traffic control devices and crossing pedestrians 3. May cause sight distance to be obscured for cross vehicles stopped to the right of the bus 4. May block the through lane during a peak period with queuing buses 5. Increases sight distance problems for crossing pedestrians
Mid-block Stop	1. Minimizes sight distance problems for vehicles and pedestrians 2. May result in passenger waiting areas experiencing less pedestrian congestion	1. Requires additional distance for no-parking restrictions 2. Encourages patrons to cross street at midblock (jaywalking) 3. Increases walking distance for patrons crossing at intersections

6.6.3 Bus Turnouts on Arterial Roadways

Providing turnouts clear of the lanes for through traffic can considerably reduce the interference between buses and other traffic. To be effective, bus turnouts should incorporate the following features:

1. A deceleration lane or taper to permit easy entrance to the loading area: This deceleration lane should be tapered 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 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 be stored in the bus turnout.
2. A standing space 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 turnout 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 turnout for a two-bus loading area of minimum design and a loading area width of 11 feet should be about 170 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.

There is an emerging trend towards bus bulbs instead of turnouts. Transit drivers are more likely to use bus bulbs or nubs than bus bays. An added bonus to using bus bulbs is that it allows for more parking and takes less right of way. Like turnouts, bus bulbs should have adequate passenger storage areas.

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.

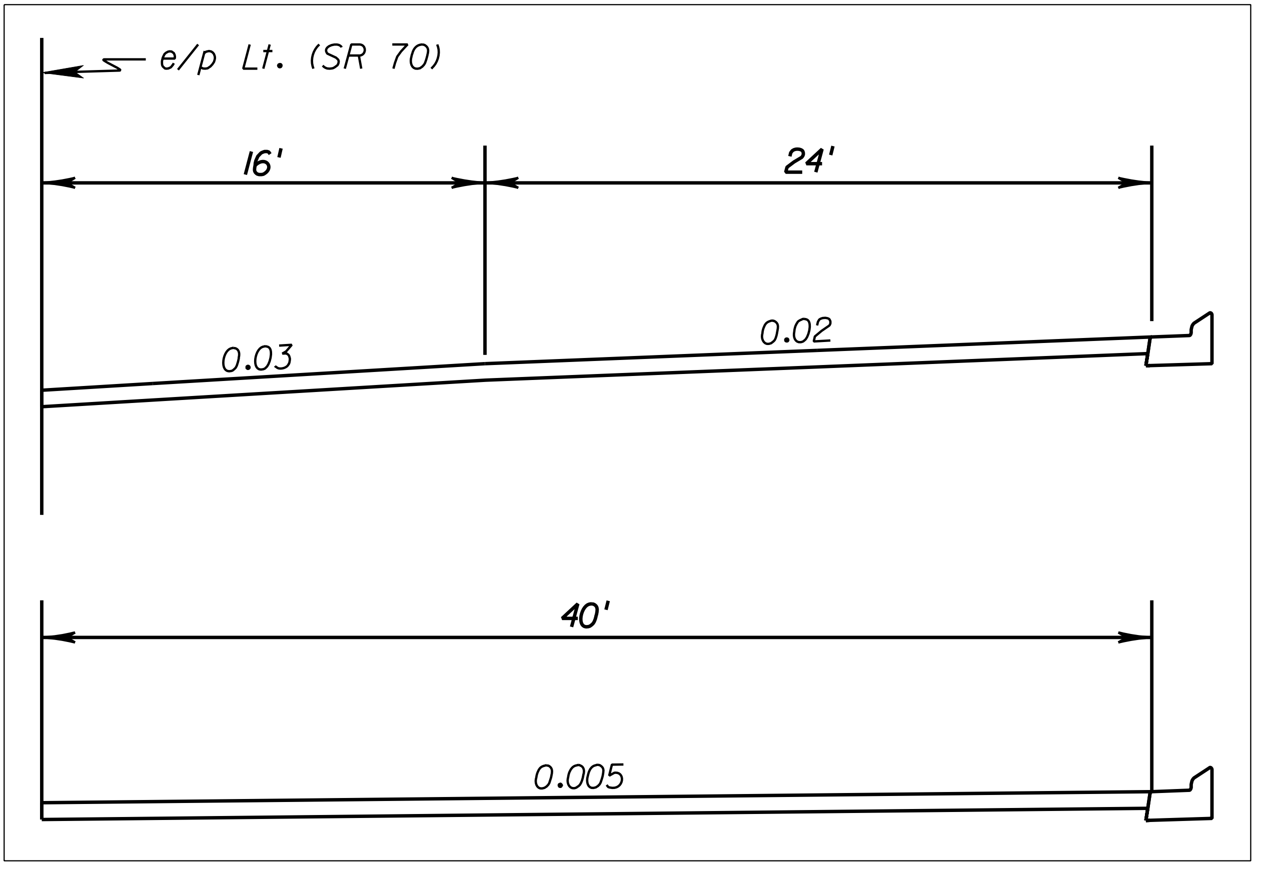
The PGL of SR 70, as shown in ***Exhibit 2***, can be adjusted vertically to best fit the project profile, accommodate the plateau transitions and maintain the roadway base clearance above the high water elevation. PIs on the profile grade line (PGL) of SR 70 are set at STA. 117+00 and STA 124+25. This 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’

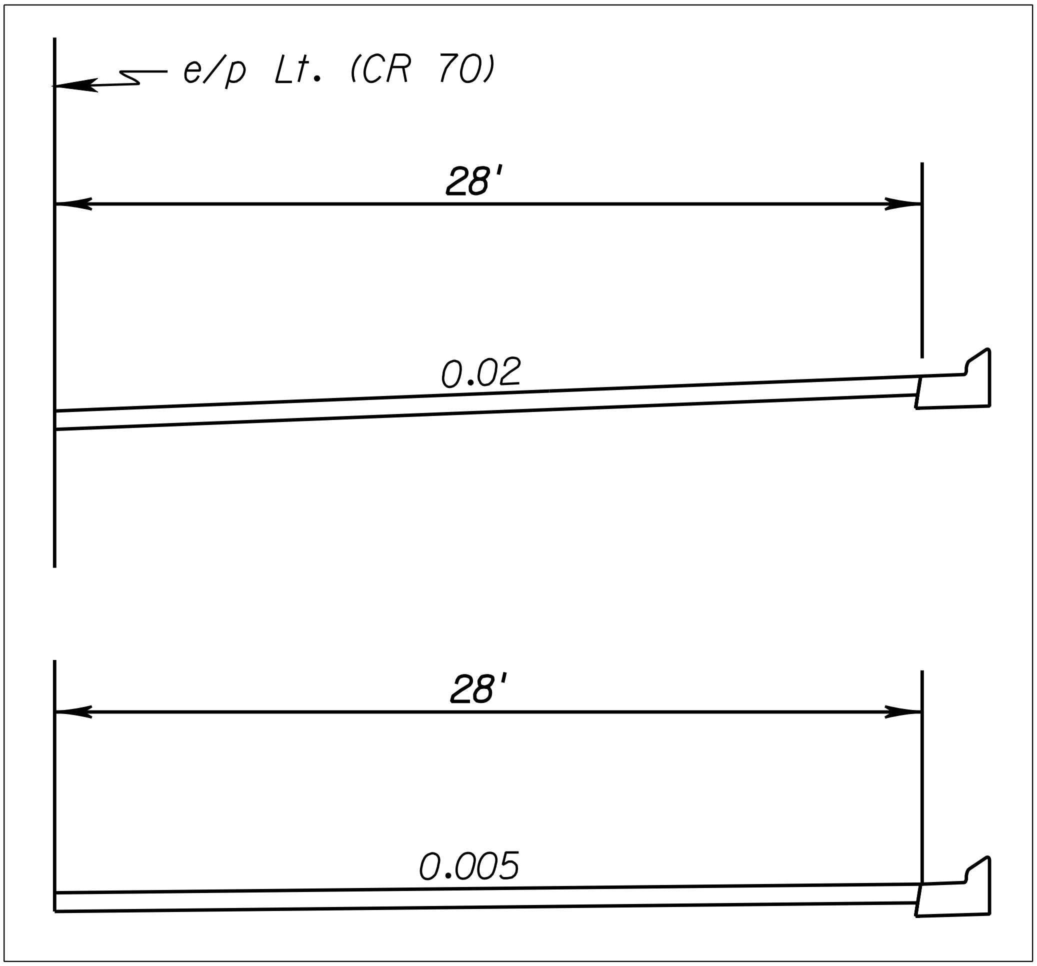
EC - NC = -0.2’ - (-0.96’) = 0.76’

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.

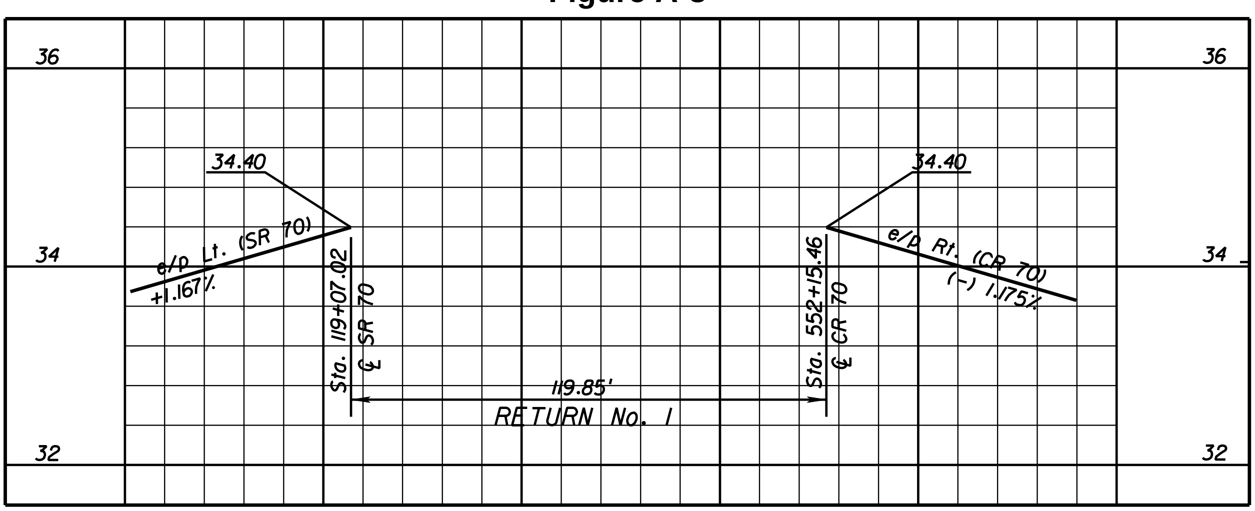
STEP 4: Calculate Elevations along the Lane Lines

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

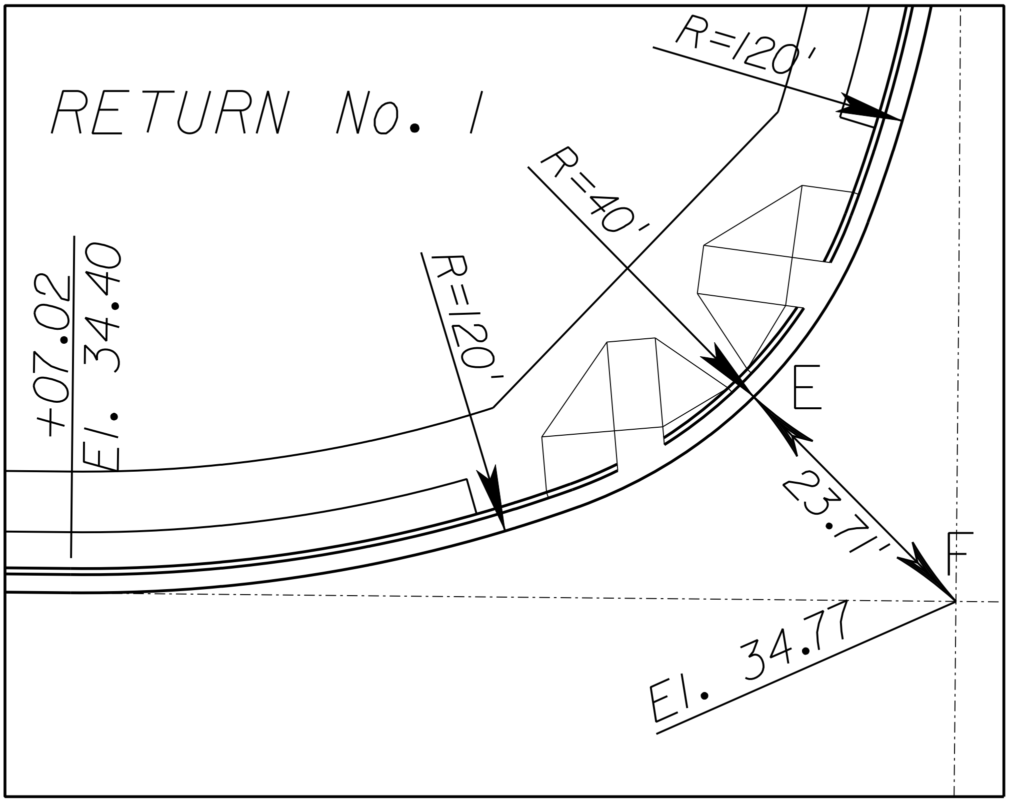
Referring to RETURN No. 1, measure the length of the return from PC to PT. Since this return is a three-centered curve it is the distance from the PC of the first curve to the PT of the third curve. Plot the return dimension on profile view and label the PC and PT stations. Next, plot the edge of pavement 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

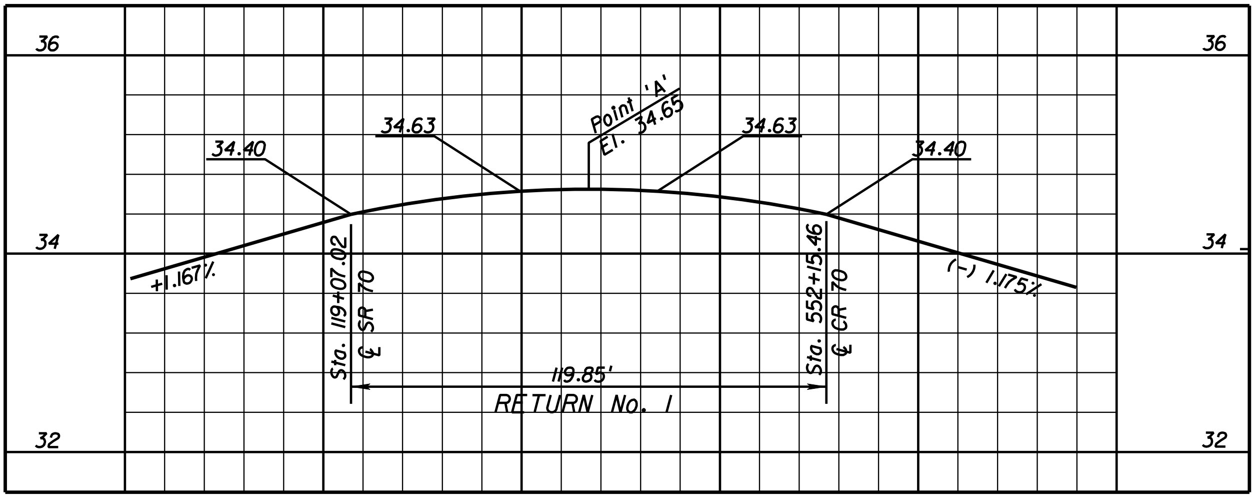


Calculate the elevation of point ‘E’ by setting a similar grade, in this case 0.5%, along vector ‘E-F’:

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.

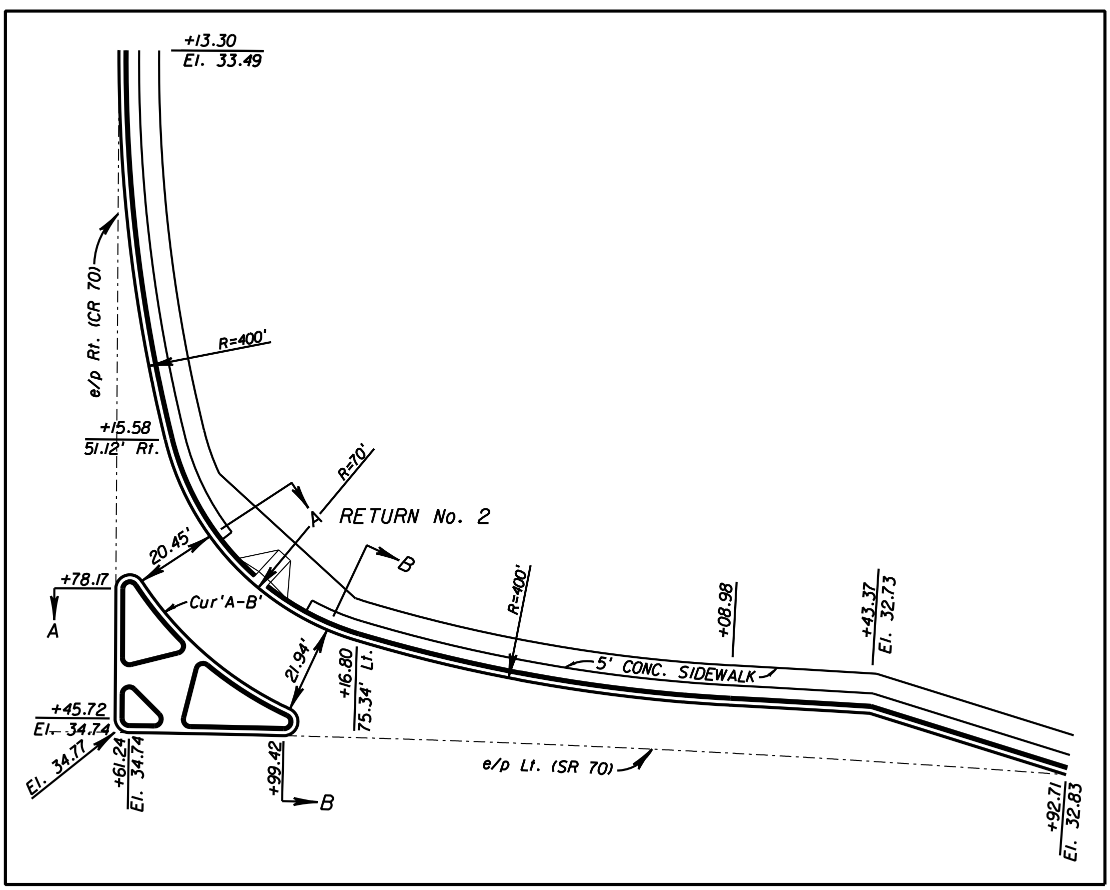
Figure A-5



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***:

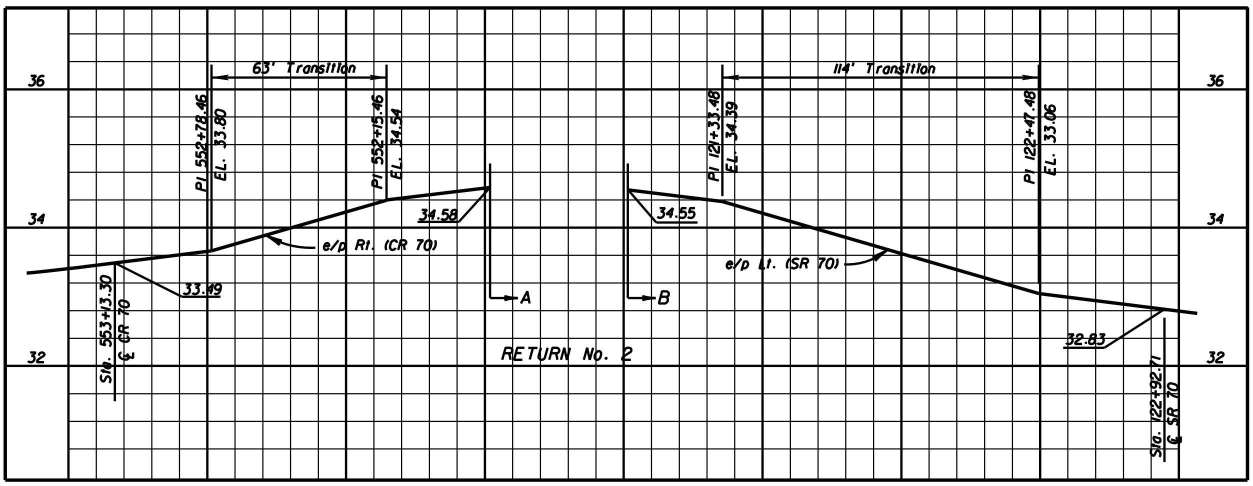
Figure A-6



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.

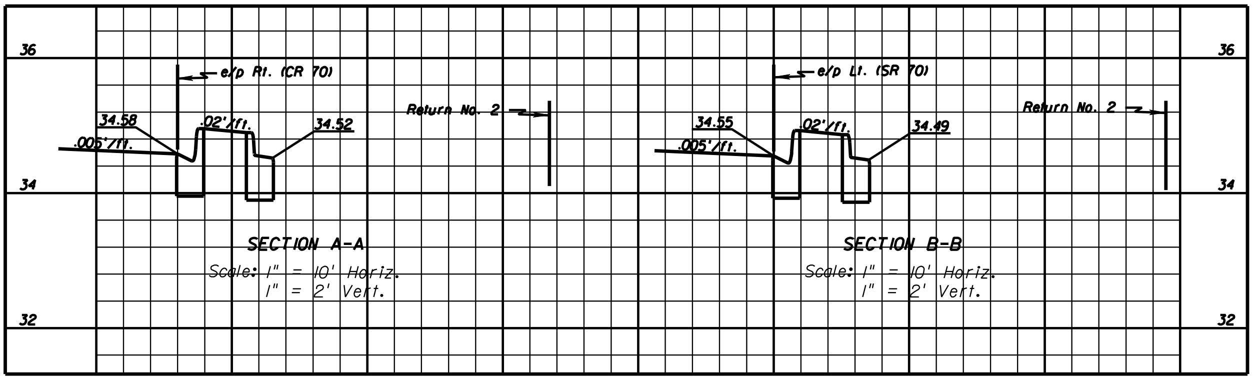
Plot the edge of pavement profiles for the two approaches from beyond the limits of the return to sections ‘A-A’ and ‘B-B’. Label the known stations and elevations as shown in ***Figure A-7***.

Figure A-7



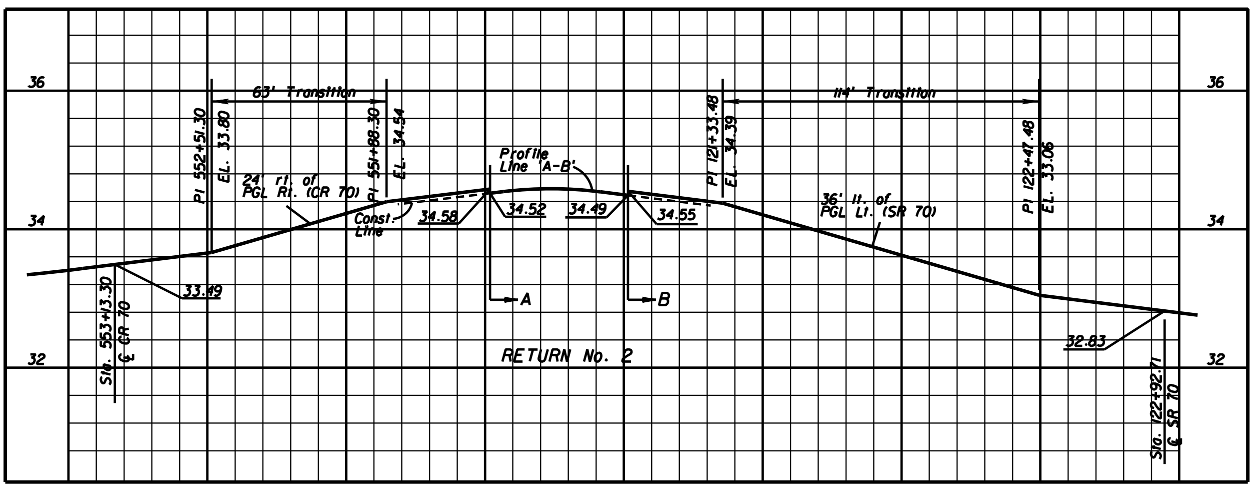
Begin plotting sections ‘A-A’ and ‘B-B’ by plotting the cross slope of the through lane and the cross section of the raised island. In this example the cross slope is 0.005’/ft. and the curb is type ‘F’. The section will go through the radius point of the island nose; therefore, the distance across the 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':

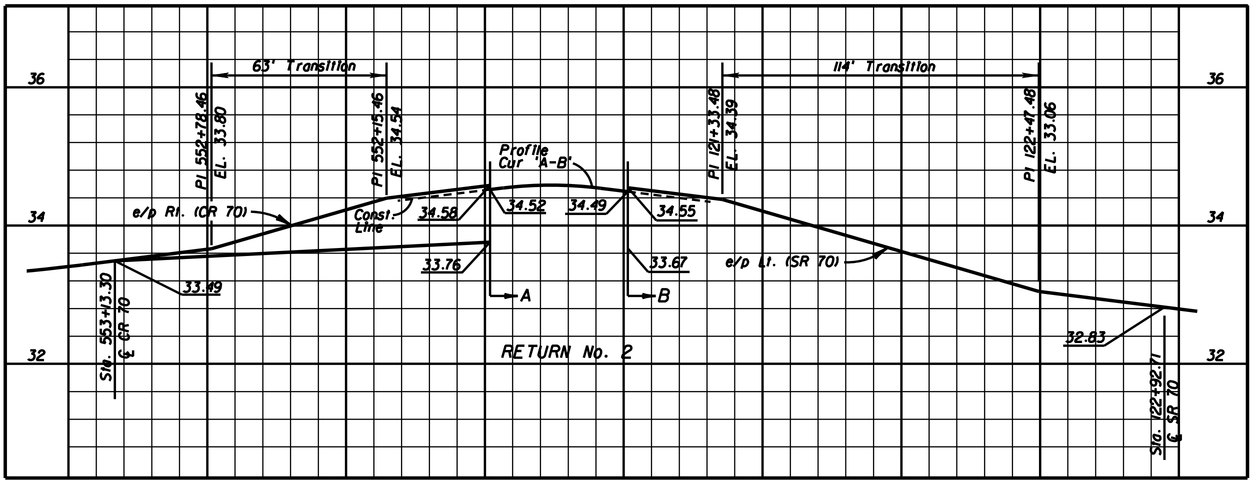
(34.52' - 33.76') / 20.45' = 0.037'/ft.

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'

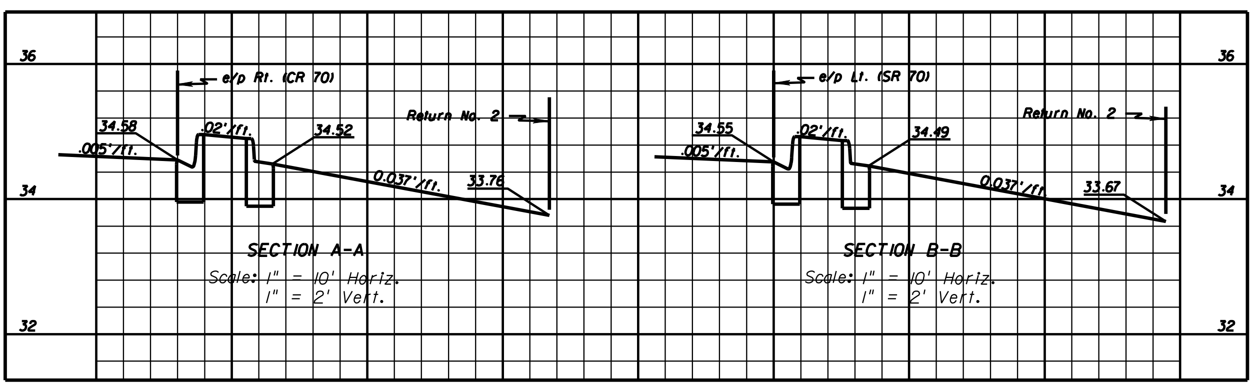
Label the elevation of the return at section 'B-B' as shown in ***Figure A-10***.

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



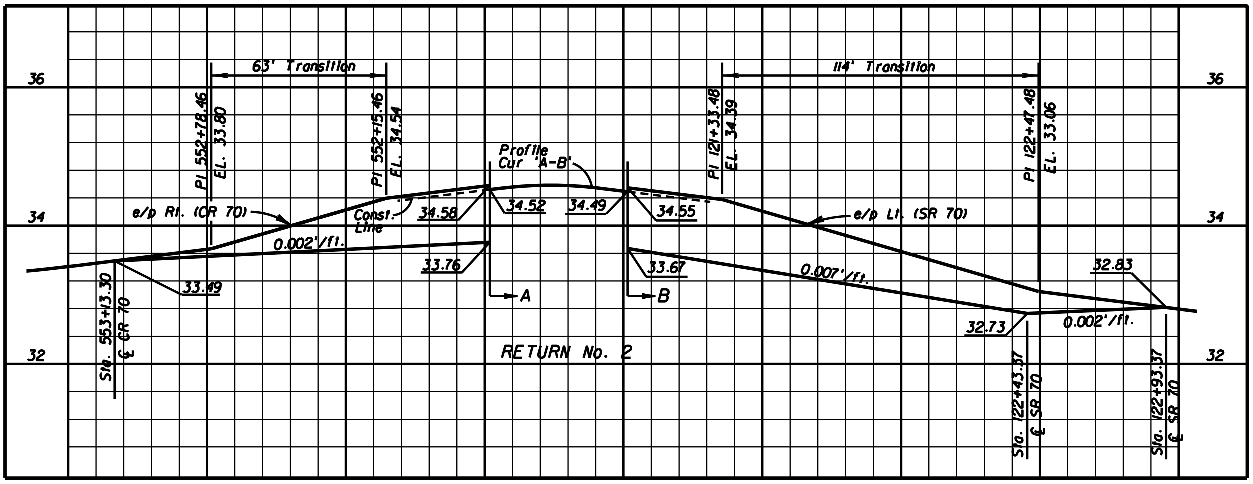
In order to avoid a flat cross slope in the vicinity of the right turn taper, it will be necessary to drain the water in the taper area back toward the intersection by setting a positive grade along the taper and installing a curb inlet at Sta. 122+43.37 Lt. Another solution would be to lengthen the 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%

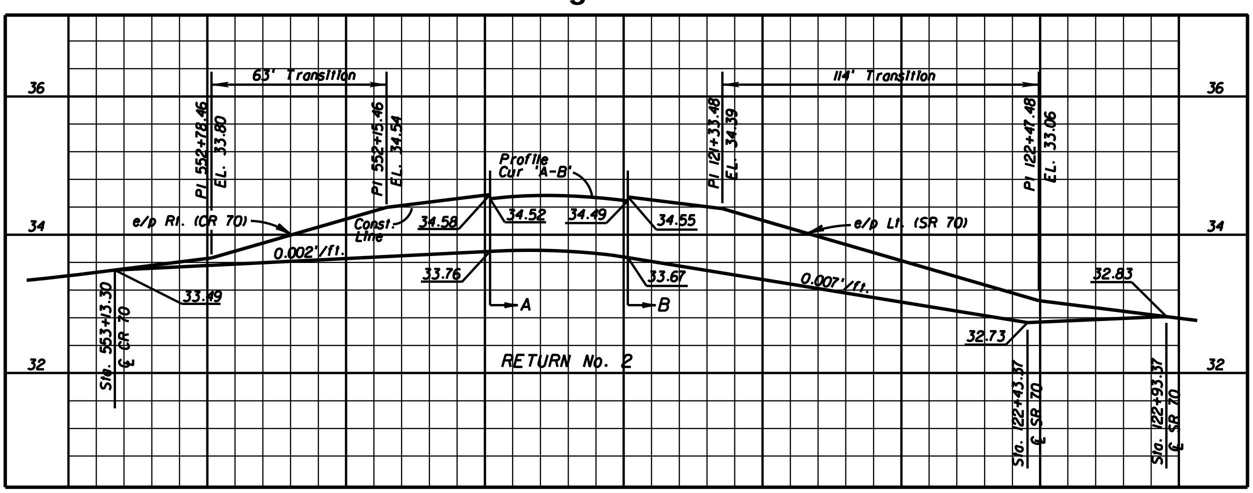
Therefore, the grade exceeds the minimum. The profile is shown in ***Figure A-12***.

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***.

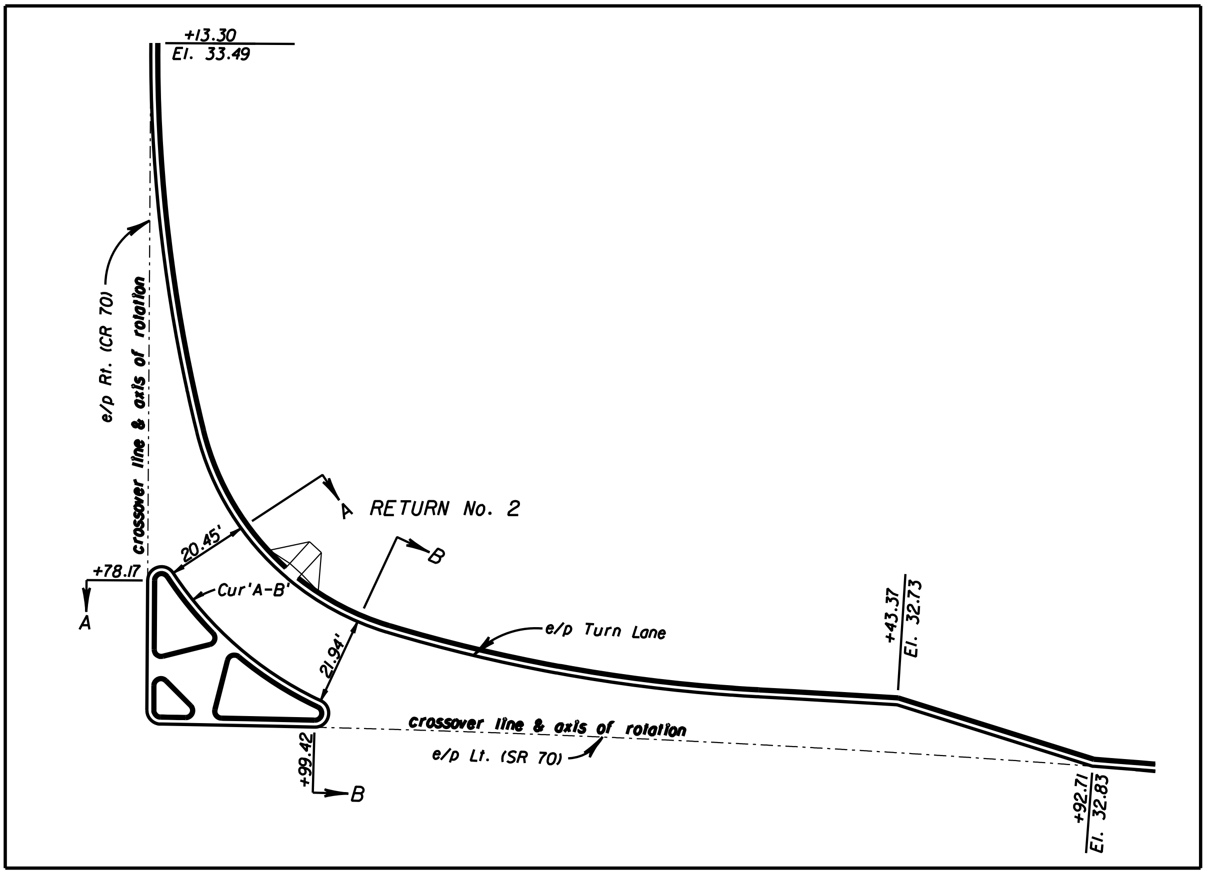
Figure A-13



STEP 7: Check the Right Turn Lane Relative Gradient and Cross Slope Break-Over

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:

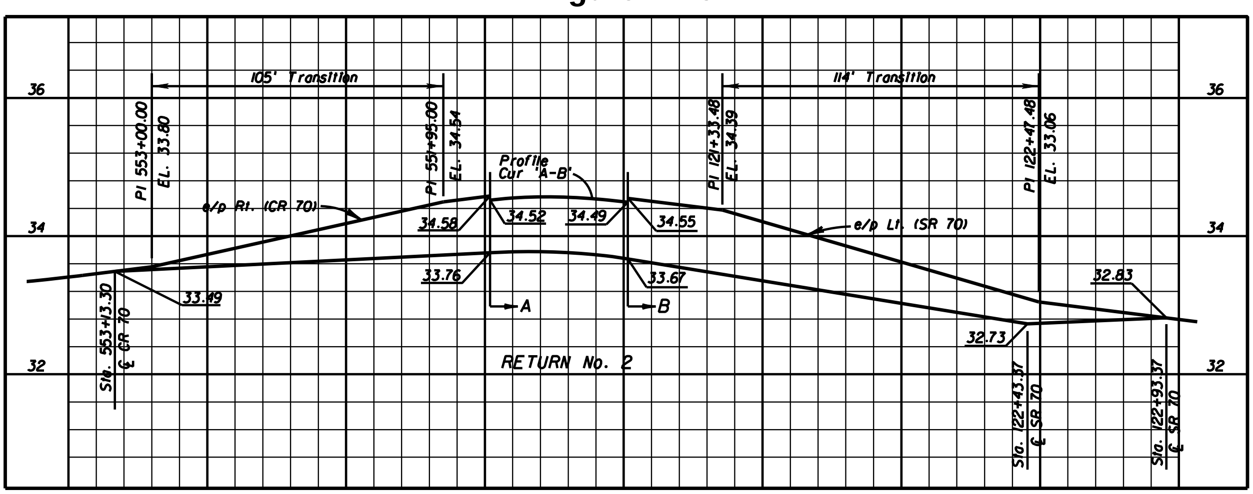
Δ = ((34.54 - 33.80)/63) x 100 - ((33.76 - 33.49)/135.13) x 100 = 0.97% > 0.54% <u>NG</u>

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***.

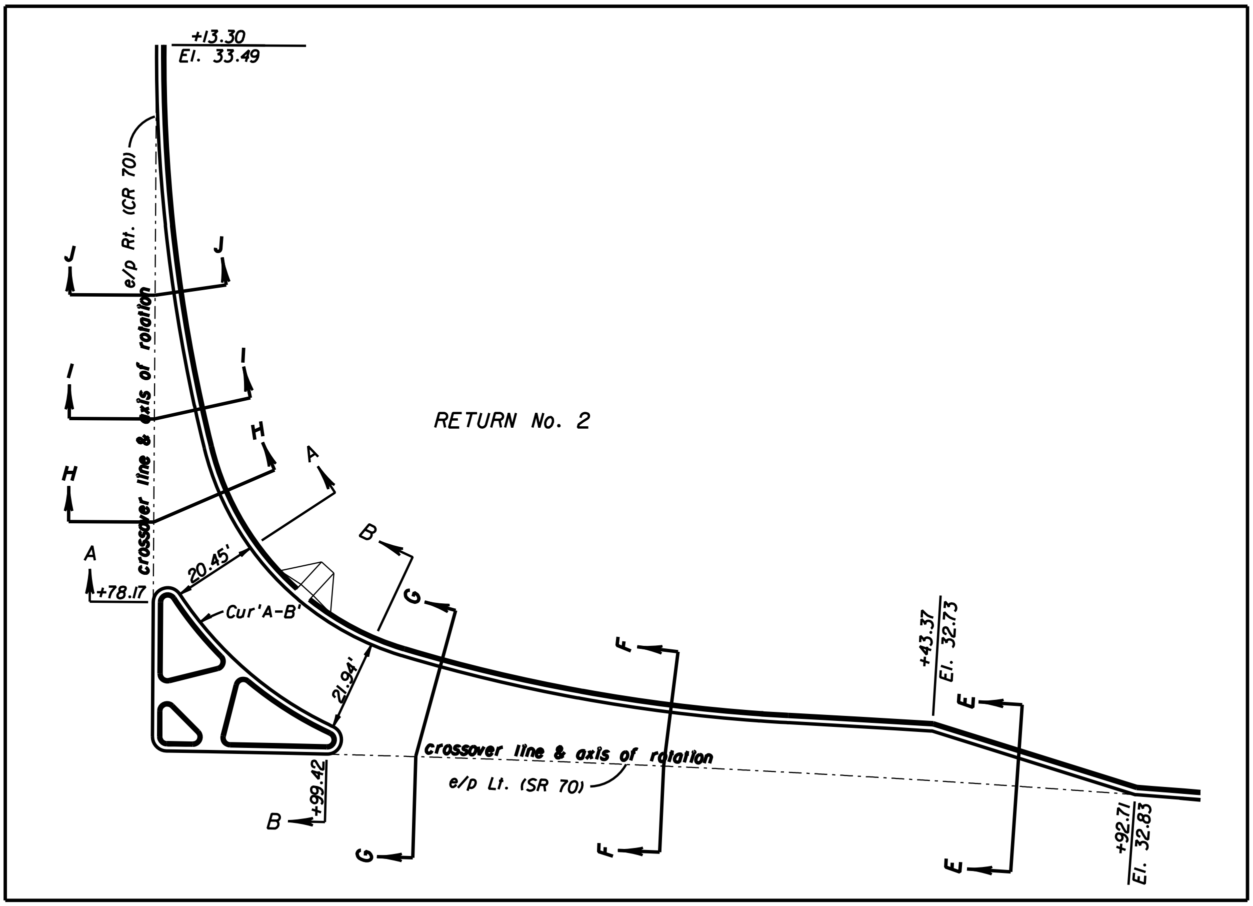
Figure A-15



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

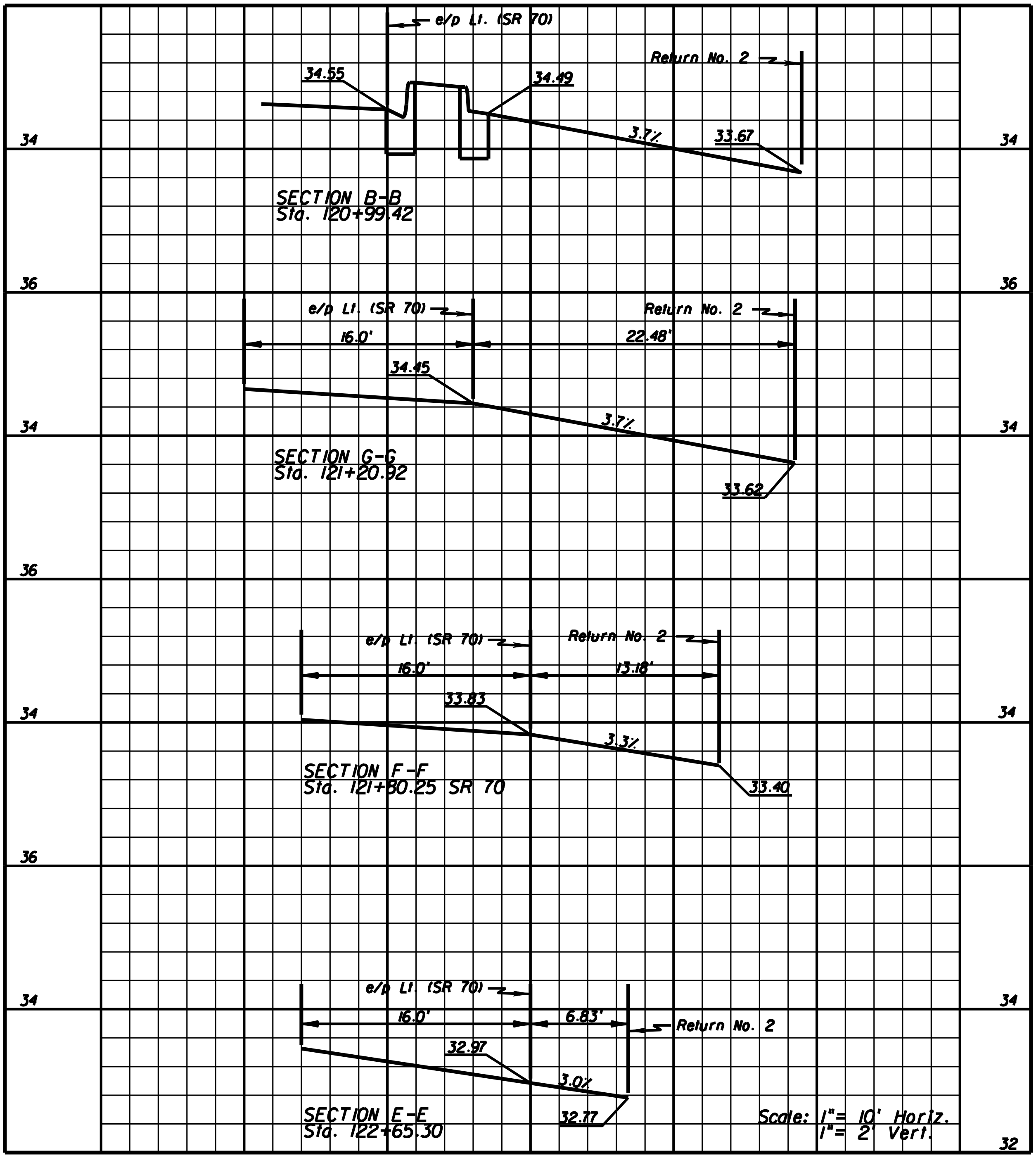
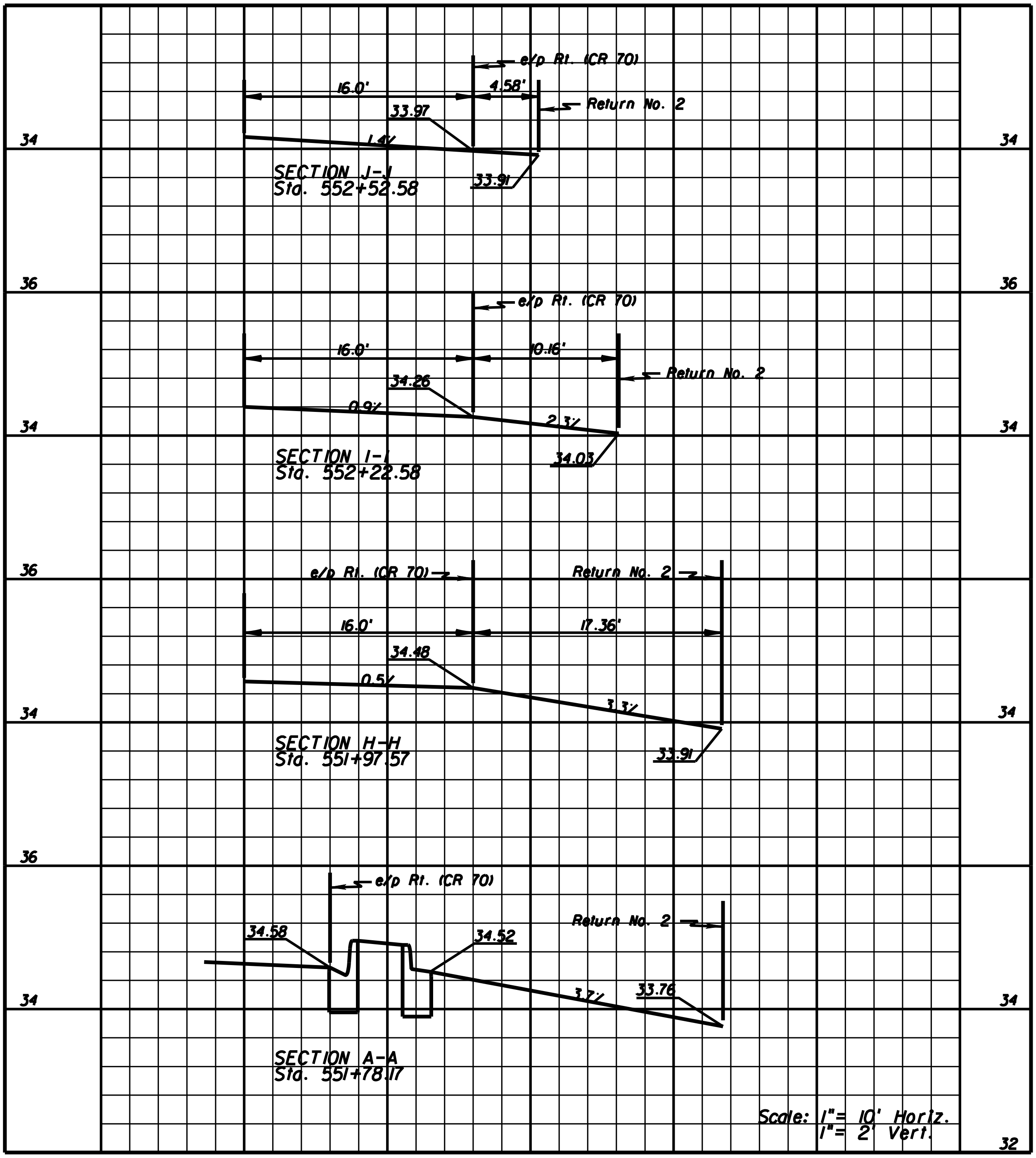
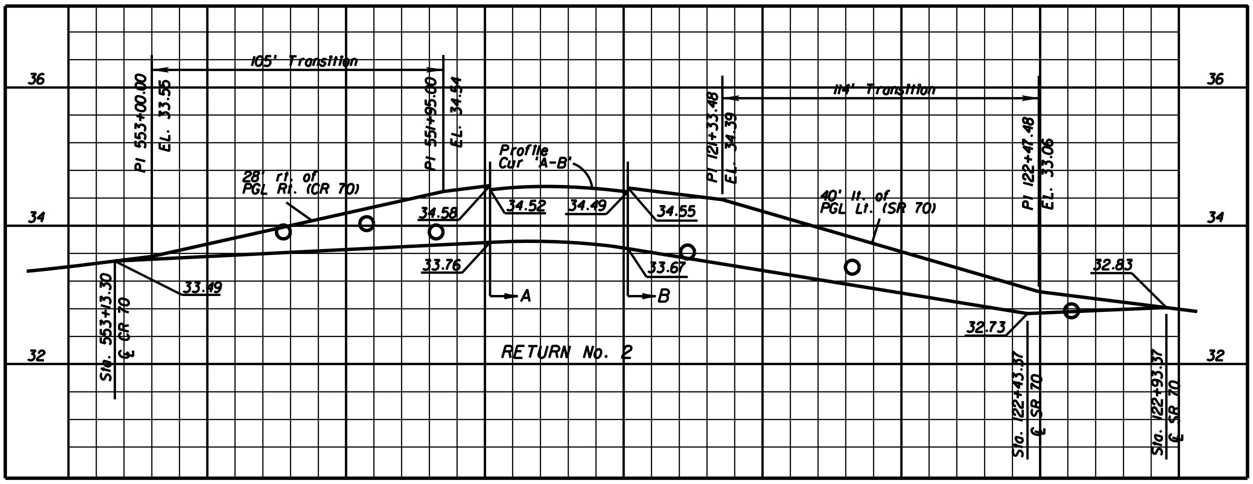


Figure A-18



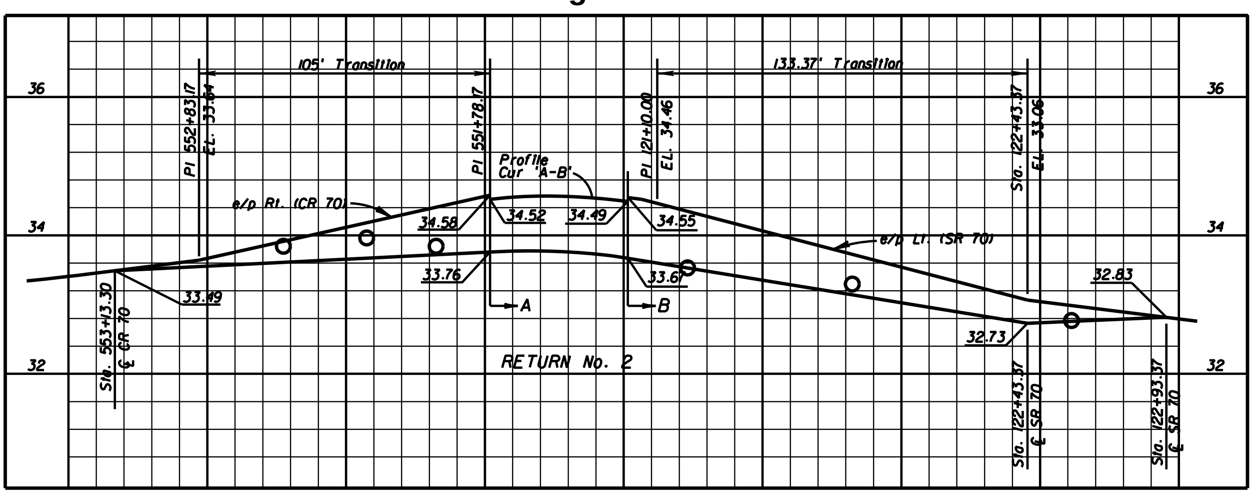
Next, plot the elevation of Return No. 2 at each section on the return profile drawing as shown in ***Figure A-19***.

Figure A-19



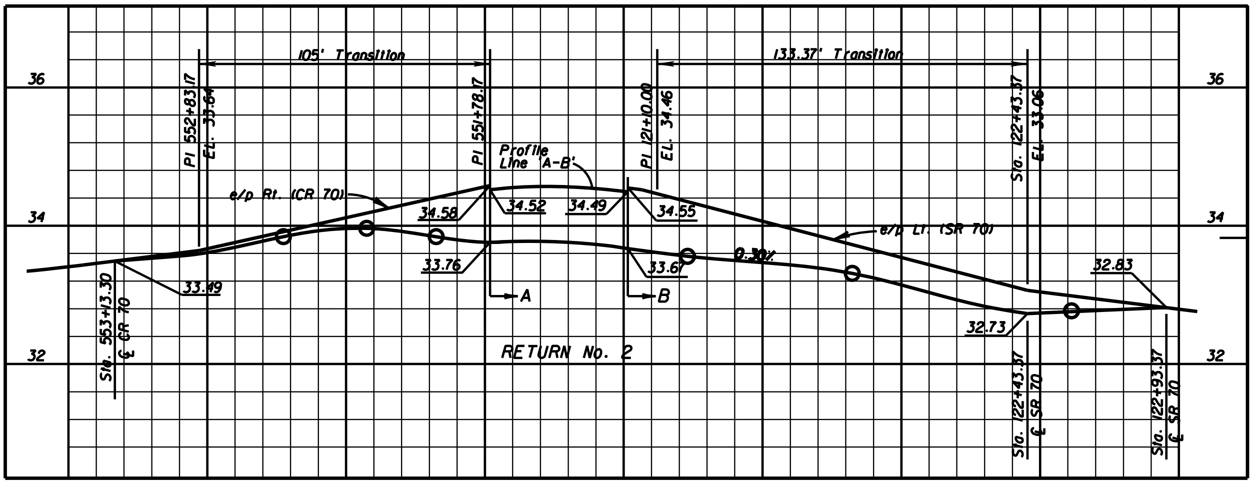
While maintaining the cross slope at each section, adjust the transitions toward the intersection in order to better match the preferred curb return profile. The entrance transition is lengthened and adjusted so that it begins at Sta. 121+10.00 and ends at Sta. 122+43.37. The return profile for the exit 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.

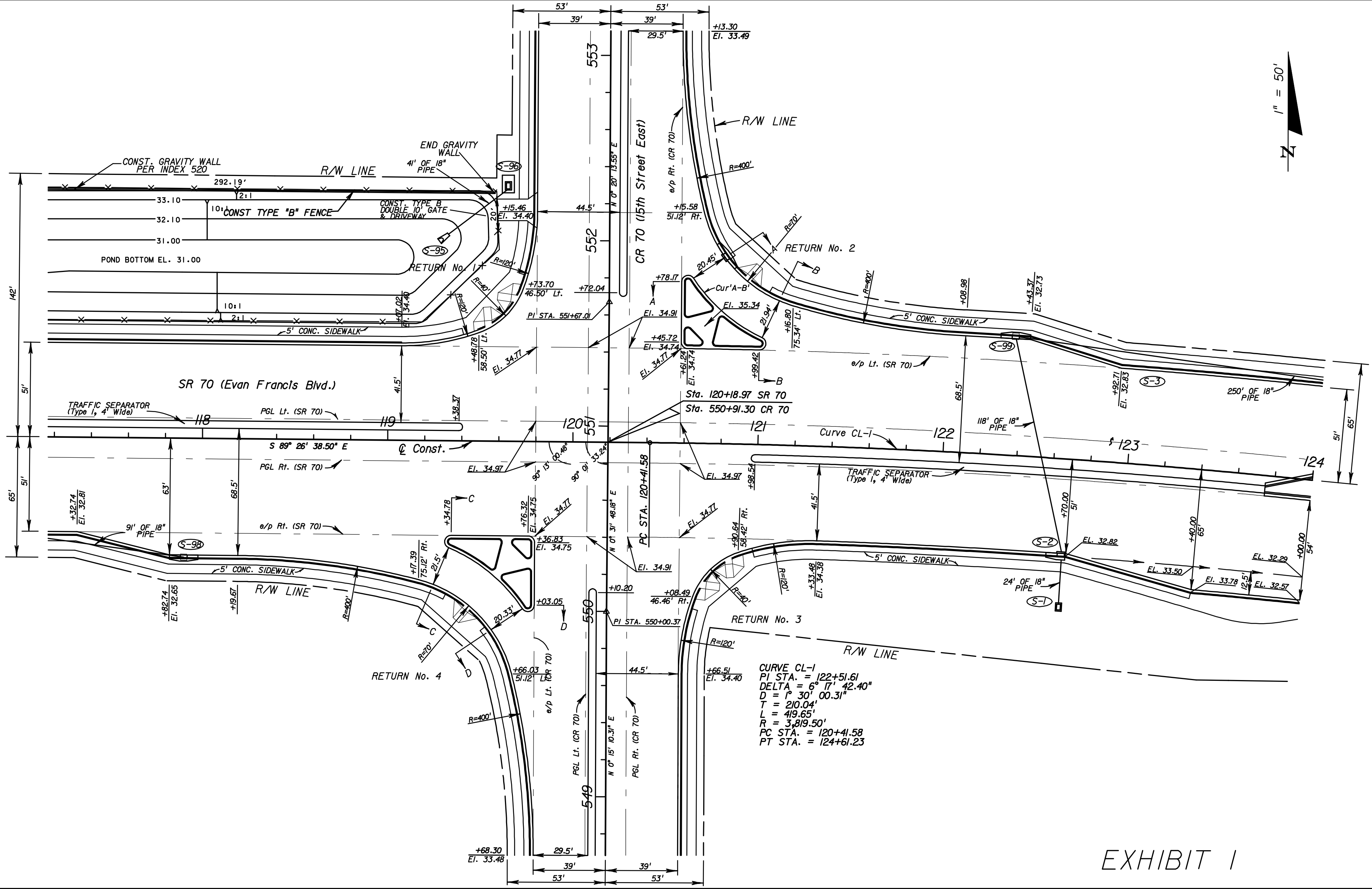


EXHIBIT I

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
70	MANATEE	196058-1-52-01

INTERSECTION DETAIL
SR 70 AT CR 70

SHEET NO.

$USER$ $DATE$ $TIME$ $FILE$

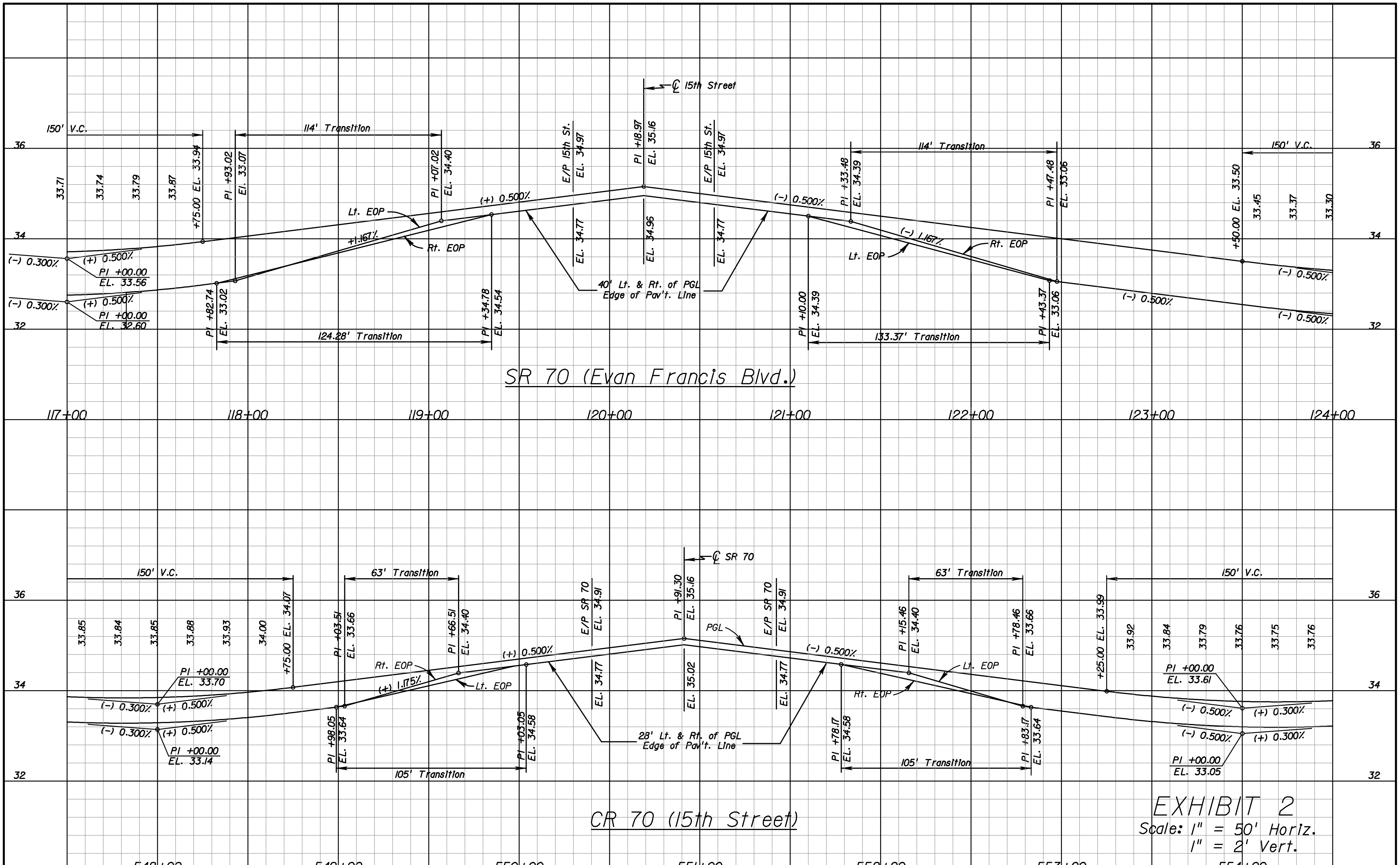


EXHIBIT 2

Scale: 1" = 50' Horiz.
1" = 2' Vert.

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
70	MANATEE	196058-1-52-01

INTERSECTION PROFILES
SR 70 at CR 70

SHEET NO.

$USER$ $DATE$ $TIME$ $FILE$

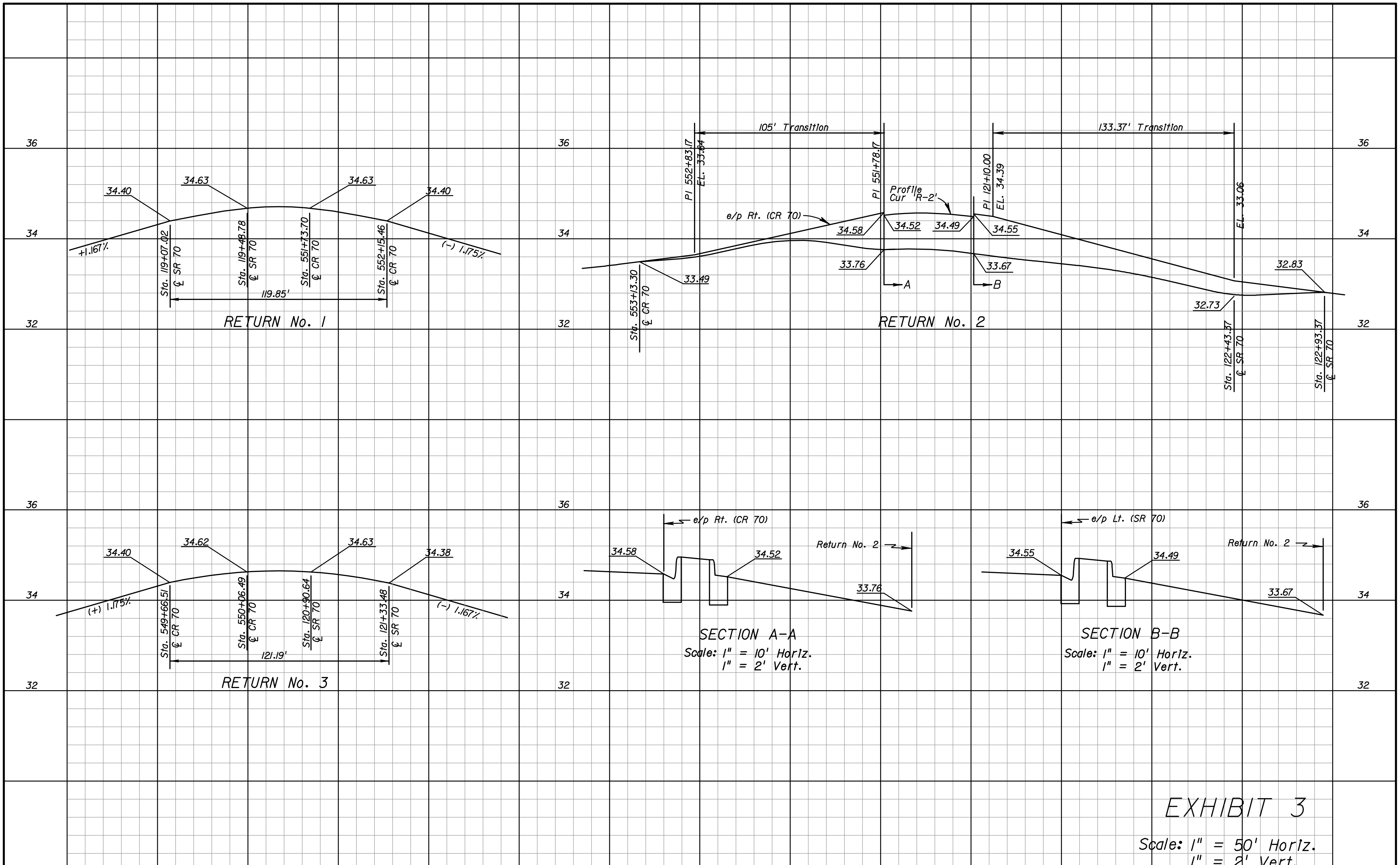


EXHIBIT 3

Scale: 1" = 50' Horiz.
1" = 2' Vert.

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

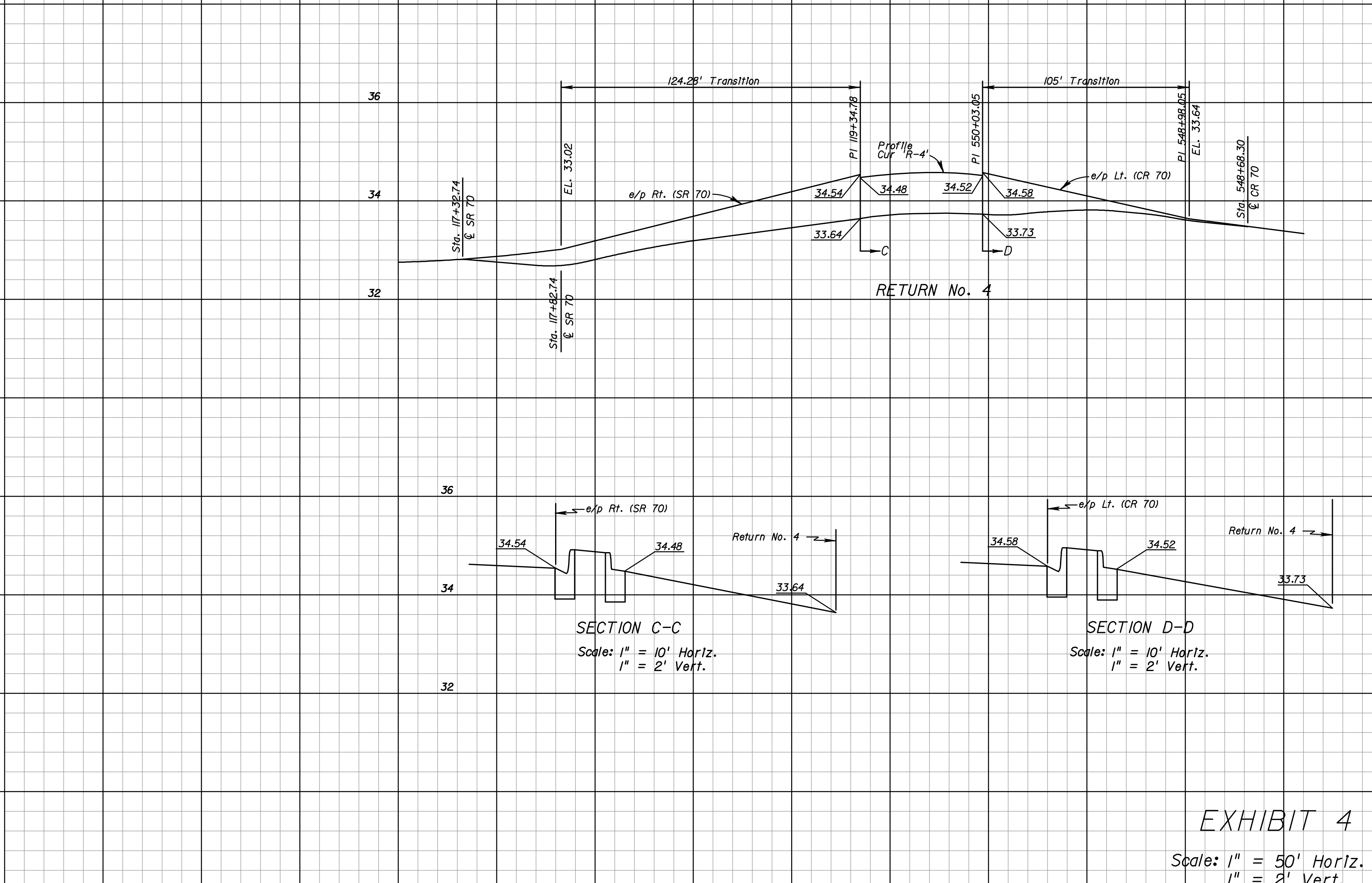
STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
70	MANATEE	196058-1-52-01

INTERSECTION PROFILES
SR 70 at CR 70

SHEET NO.

$USER$ $DATE$ $TIME$ $FILE$



REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
70	MANATEE	196058-1-52-01

INTERSECTION PROFILES
SR 70 at CR 70

SHEET NO.

$USER$ $DATE$ $TIME$ $FILE$