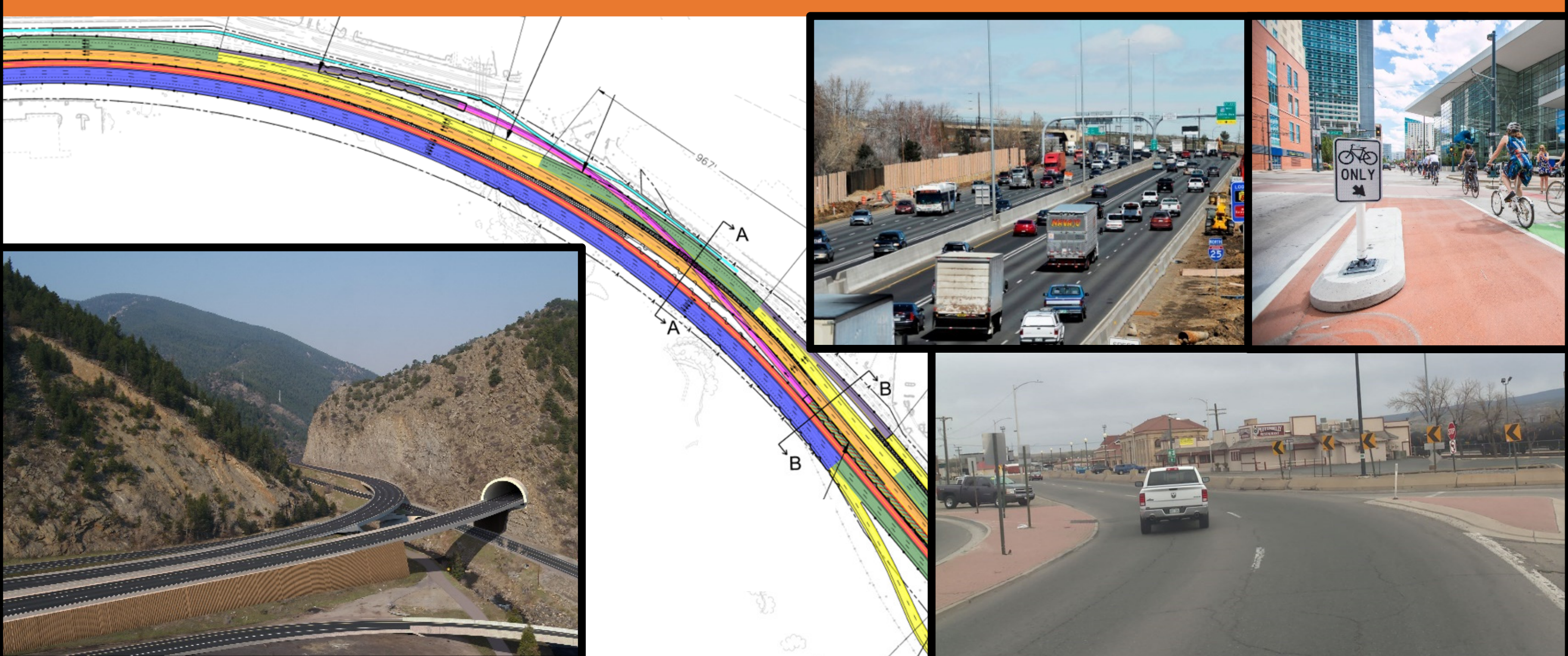
COLORADO
Department of Transportation
Division of Project Support

2018
Roadway Design Guide



ACKNOWLEDGEMENTS

CDOT Regional Representatives

REGION 1
Adam Parks

REGION 2
Cynthia Bailey
Shane Ferguson

REGION 3
Kaitlyn Clark
Jacob Rivera

REGION 4
Richard Christy

REGION 5
Jennifer Allison
Jason Fullerton

Headquarters
Sahar Alola
Louis Avgeris
Ken Brubaker
Verlen (Mike) Coy
Jerome Estes
Scott Hoftiezer
Alex Karami
Joshua Keith
Neil Lacey
Andrew Pott
David Swenka

Stanley Consultants
Jeff Binning
Sarah Zarzecki

INTRODUCTION

The 2018 Colorado Department of Transportation (CDOT) Roadway Design Guide is an update to the 2005 CDOT Roadway Design Guide based on the 2011 edition of the American Association of State Highway and Transportation Officials' A Policy on Geometric Design of Highways and Streets (PGDHS), commonly known as the AASHTO Green Book, and upon current CDOT policies and practices. Although this guide is based on the PGDHS, not every section of the PGDHS is represented in the guide. When a more detailed discussion of theory or background is needed, the designer should refer to the appropriate section of the PGDHS.

A blended team comprised of roadway design consultants and the design program manager from the Project Development Branch prepared the 2018 updates to the 2005 CDOT Roadway Design Guide. The 2018 updates were reviewed and commented upon by project delivery representatives from all CDOT regions, TSM&O and the Division of Transportation Development (DTD).

Consult Chapter 3, Elements of Design, and Chapter 4, Cross Section Elements, for details on the basic design elements applicable to all classifications of roadways.

Use these guidelines and engineering judgment, along with the PGDHS. Users should keep abreast of the latest practices and developments in design. These guidelines are not intended to be a detailed design manual that supersedes the need for the application of sound principles by the knowledgeable design professional. Designers are permitted the flexibility and encouraged to develop designs tailored to particular situations when appropriate. Minimum values are either provided or implied by the lower value in a given range of values. Variations from minimum design criteria should be thoroughly evaluated and documented with a Design Exception Variance Request or a design decision, as outlined in the CDOT Project Development Manual.

The Engineer of Record is responsible for incorporating current design standards and safety guidelines as developed by AASHTO and CDOT into highway design.

The 2018 CDOT Design Guide:

- Refers to The American Association of State Highway and Transportation Officials (AASHTO) *A Policy on Geometric Design of Highways and Streets 2011* as the *"PGDHS."*
- Clarifies and, as needed, expands on material in the *PGDHS.*
- Points out processes and applications unique to CDOT.
- Refers to other resources where appropriate, e.g. the *Highway Capacity Manual.*

Comments on this Guide may be sent to:

Colorado Department of Transportation
Standards and Specifications Unit
2829 W. Howard Place
Denver, Colorado 80204
Or the design program manager.

These guidelines use standard US Customary (inch-feet-mile) units.

CHAPTER 1
FUNCTIONAL AND CONTEXTUAL HIGHWAY CLASSIFICATIONS

List of Tables

List of Figures

CHAPTER 2
DESIGN CONTROLS AND CRITERIA

List of Tables

List of Figures

CHAPTER 3
ELEMENTS OF DESIGN

List of Tables

List of Figures

CHAPTER 4
CROSS SECTION ELEMENTS

List of Tables

List of Figures

CHAPTER 5
LOCAL ROADS AND STREETS

List of Tables

CHAPTER 6
COLLECTOR ROADS AND STREETS

CHAPTER 7
RURAL AND URBAN ARTERIALS

List of Tables

CHAPTER 8
FREEWAYS

CHAPTER 9
INTERSECTIONS

List of Tables

List of Figures

CHAPTER 10
GRADE SEPARATIONS AND INTERCHANGES

List of Tables

List of Figures

CHAPTER 11
ACCESS CONTROL AND ACCESS MANAGEMENT

List of Tables

List of Figures

CHAPTER 12

ACCESSIBLE PEDESTRIAN DESIGN

List of Figures

CHAPTER 13
ALTERNATE STANDARDS (LOW VOLUME ROADS)

List of Tables

CHAPTER 14
BICYCLE AND PEDESTRIAN FACILITIES

List of Figures

List of Tables

CHAPTER 15
BRIDGE

CHAPTER 16
CONSTRUCTION SPECIFICATIONS

List of Tables

List of Figures

CHAPTER 17

This chapter is currently under development.

CHAPTER 18
NOISE GUIDE

List of Tables

List of Illustrations

CHAPTER 19
ROUNDABOUTS

List of Tables

List of Figures

CHAPTER 20
TRAFFIC AND SAFETY ENGINEERING

CHAPTER 1
FUNCTIONAL AND CONTEXTUAL HIGHWAY CLASSIFICATIONS

1.0 INTRODUCTION

The function of a highway is to provide a facility that allows the movement of people, goods, and services. Different types of facilities are needed for various vehicle travel movements. To differentiate between the types of highways, functional and contextual classifications have been developed so that engineers, administrators, and the general public can communicate about highways. A functional classification is a roadway descriptor determining hierarchal status within the transportation network and the expectational service provided, whereas a context classification system provides guidance regarding appropriate design accommodation using the characteristics of the roadway's surrounding environment. The state of the practice is supplementing the functional classification system with a process to consider context and user needs. By determining the functional and context classifications, designers can have a better understanding of the criteria necessary to accommodate user's needs and safety. The *2018 Roadway Design Guide's* primary focus for standards and roadway considerations is based on Functional Classifications. However, the contextual framework provided in *NCHRP Report 855* **(3)** can assist in making contextually appropriate design decisions.

1.1 FUNCTIONAL CLASSIFICATIONS

Definitions of highway facilities in urban and rural settings are described below and provide additional information on the range of mobility and access each classification serves. Ramps and other non-mainline roadways should be assigned the functional classification of the highest functional classification of the intersecting mainline that serves the ramp or other facility. The functional classifications exist in both the urban and rural areas. The functional classifications are:

- Arterials
  - Principal Arterials. Principal Arterials serve a large percentage of travel between cities and activity centers. Principal arterials are typically roadways with high traffic volumes and are the frequent route for intercity busses and trucks. Principal Arterials provide a high degree of mobility and carry a high percentage of travel for long distance trips including those that go directly through or bypass activity centers. Principal Arterials are stratified into three classifications:
    - Interstate. Interstates are the highest classification of Arterials and are designed and constructed for mobility and long-distance travel. Roadways in this category are officially designated as Interstate by the Secretary of Transportation and are considered Principal Arterials.
    - Other Freeways & Expressways. Roadways in this category with full access control look similar to Interstates. By definition, Freeways are characterized by full access control with access points limited to on/off ramps and no at grade intersections. Expressways are more common in rural settings where at grade intersections are permitted to varying degrees depending on context. In general, these types of roadways favor mobility over access with this being truer for Freeways than Expressways.

        - Other Principal Arterials. These roadways serve activity centers and provide a high degree of mobility. These roadways provide additional access to parcels and have at-grade intersections. Other Principal Arterials provide similar service in both urban and rural areas; the primary difference in urban areas is the quantity of arterials serving a particular urban area and radiate out from the urban center. Rural areas would typically be served by one Arterial.

    - Minor Arterials. Minor Arterials provide service for moderate length trips, serve geographic areas that are smaller than the Principal Arterial roadways, and have higher connectivity to the Principal Arterials. In urban settings, they interconnect and supplement the Principal Arterial system, connect communities, and may carry many bus routes. In rural settings, they are typically designed to provide higher travel speeds with minimum interference to the trough movement.

- Collector Roads and Streets. Collectors provide the connection from Local Roads to the Arterial systems. Collectors may be subdivided into Major and Minor Collectors in both the urban and rural areas. A major part of the rural highway system consists of two-lane collector highways. The rural collector routes generally serve travel of primarily intra-county rather than statewide importance and constitute those routes on which predominant travel distances are shorter than on arterial routes. An urban collector street is a public facility that includes the entire area within the right of way. The urban collector street also serves pedestrian and bicycle traffic and often accommodates public utility facilities within the right of way.

    - Major Collector. Major Collectors are typically longer in length, have lower connecting driveway density, higher posted speeds, higher traffic volumes and may have more travel lanes than the Minor Collector.
    - Minor Collector. Minor Collectors serve both land access and traffic circulation, penetrate residential neighborhoods for short distances, operate at lower posted speeds, and have signalized intersections, provide service to smaller communities not served by Arterials, and link locally important traffic generators with rural surroundings.

- Local Roads and Streets. Local Roads account for the largest percentage of roadways in terms of mileage and are typically designed to discourage through traffic. A local road or residential street primarily serves as access to a farm, residence, business, or other abutting property. Some such roads properly include geometric design and traffic control features more typical of Collectors and Arterials to encourage the safe movement of through traffic. On these roads, the through traffic is local in nature and extent rather than regional, intrastate, or interstate. Local Roads are typically classified by default; once all other roads have been classified as Arterial or Collector, the remainder are Local Roads.

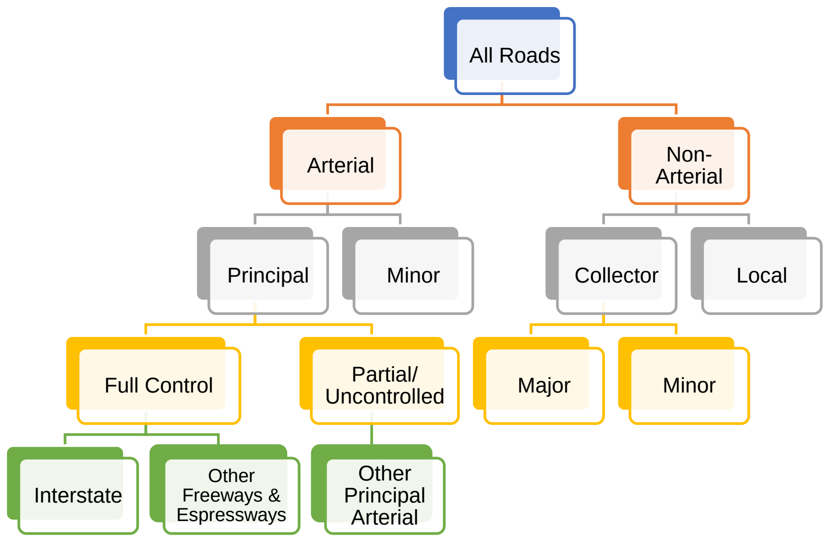


Figure 1-1 (Figure 3-4 of Highway Functional Classification Concepts, Criteria and Procedures (5)) Federal Functional Classification Decision Tree

Functional classification determination for urban and rural designation is tied to the census, which is taken every ten years. Following the census, the urban boundaries are realigned. That realignment mostly affects off-system roads. Because there is a set percentage of roadway that can be on the National Highway System (NHS), new roadways are seldom added. Sometimes there are requests to change the functional classification, such as the rare instance in which significant construction has upgraded a collector to an arterial or a two lane is made into a four or six lane. The planning agencies –Transportation Planning Regions and Metropolitan Planning Organizations (TPRs and MPOs) – work with CDOT, which then takes the request to the Transportation Commission and the Federal Highway Administration (FHWA). Classification changes are made prior to design.

The key factor the designer needs to determine is whether the project is on a federal-aid highway, and if so, where the project lies on the system. The *CDOT Online Transportation Information System (OTIS)* **(1)** website contains the CDOT GIS data map.

If funding has been accepted, a construction contract is awarded, or a project is underway, the functional classification cannot be changed.

The basic concepts required for understanding the functional classification of highway facilities and systems are discussed at length in Chapter 1 of the *PGDHS* **(2)**.

1.2 CONTEXT CLASSIFICATIONS

Contextual settings can be used to identify distinctions which require different practices or approaches during design. The general context considerations include density, land use, and building setback. The Context Classifications system may be used in combination with the Functional Classifications to provide a matrix approach to design based on speed, mobility, access, and user safety/needs. The distinct context settings are:

- Rural. The rural context range includes areas of no to little development, commonly used for farming, mineral extraction, or recreation. Very few residential or commercial structures exist. Pedestrians are few, bicycles are likely used for recreational purposes and not commuter, and transit is limited. Building setbacks are generally large.
- Rural Town. The rural town context includes areas of low density, but with some concentration of development. Rural towns are usually incorporated, but may have limited services. Rural towns are comprised of both residential and commercial structures. Rural towns may have a main street corridor, and commonly have on-street parking and sidewalks. Pedestrians and bicycle commuters are common, but transit is limited. Building setbacks are generally small. Roadways must accommodate through travelers as well as residents.
- Suburban. The suburban context includes areas of medium density usually of residential, commercial, and light industrial structures. These structures usually accommodate off-street parking. In most instances, sidewalks are present and sometimes bike lanes. Multi-use developments may include town centers and multi-family housing. Suburban settings are usually around urban areas. Pedestrians, bicyclists, and some transit may be common, however; the dominant mode of travel is by passenger vehicles. Building setbacks vary.
- Urban. The urban context includes areas of high density usually of residential, commercial, and light and heavy industrial multi story structures. Urban areas host prominent destinations including sporting or conference venues. Parking is usually both on and off-street on surface lots or parking structures. Pedestrian and bicycle use is high and transit options are abundant. Building setbacks are varied.
- Urban Core. The urban core context includes areas of the highest density usually of residential and commercial high-rise structures. The urban core is the business center of a metropolitan area. Parking is either on-street and time restricted, or off-street in parking structures. Pedestrian and bicycle use is high and transit options are abundant. Building setbacks are usually less than in the urban context.

Table 1.1 illustrates characteristics of each context category. See the *NCHRP Report 855* **(3)** for additional context classification information.

Category	Density	Land Use	Setback
Rural	Lowest (few houses or other structures)	Agricultural, natural resource preservation and outdoor recreation uses with some isolated residential and commercial	Usually large setbacks
Rural Town	Low to medium (single family houses and other single purpose structures)	Primarily commercial uses along a main street (some adjacent single family residential)	On-street parking and sidewalks with predominantly small setbacks
Suburban	Low to medium (single and multi-family structures and multi-story commercial)	Mixed residential neighborhood and commercial clusters (includes town centers, commercial corridors, big box commercial and light industrial)	Varied setbacks and mostly off-street parking
Urban	High (multi-story, low rise structures with designated off-street parking)	Mixed residential and commercial uses, with some intuitional and industrial and prominent destinations	On-street parking and setbacks with mixed setbacks
Urban Core	Highest (multi-story and high-rise structures)	Mixed commercial, residential and institutional uses within and among predominately high-rise structures	Small setbacks with sidewalks and pedestrian plazas

Table 1-1 (Table 1 of the *NCHRP Report 855*) Context Category Descriptions

1.3 ROADWAY USERS

The primary categories of roadway users and associated design considerations are:

- Automobiles. Automobile design accommodations include operating speed, mobility, and accessibility. Speed and mobility are the highest in a rural context and lowest in an urban core context. The opposite is true for accessibility. Similarly, speed and mobility are highest in a higher functional classification whereas access decreases for higher functional classifications. Automobiles consist of passenger cars, motorcycles, and light trucks.
- Bicyclists. Bicycle design accommodation is generally categorized based on the level of separation provided from motor vehicle traffic. Bicycle network functional classifications include citywide connectors, neighborhood connectors, and local connectors. For higher order bicycle routes, or higher order roadways, more separation from traffic is generally recommended (see the *NCHRP Report 855* **(3)** for additional information).

- Pedestrians. Pedestrian design accommodation is categorized primarily using the width of the facility provided. Pedestrian facilities can be divided into four categories which are based on the expected level of use. When increased pedestrian volumes are expected, wider facilities should be provided (see the *NCHRP Report 855* **(3)** for additional information).

1.4 PERFORMANCE BASED PRACTICAL DESIGN

Performance based practical design (PBPD) is a process in which design decisions are data driven from performance analysis. The quantitative performance analysis guides decision-making throughout the project development process. Ideally, stakeholder consensus is achieved on the desired project performance outcomes (goals) in the planning phase. Using these goals as a guide through project development allows for greater focus on optimizing system level needs while meeting clearly defined project level needs. For example, a PBPD opportunity may present itself in the form of a scenario where meeting full design standards for a geometric element comes at significant cost to the system without adding notable performance value relevant to the project's goals.

With many sample intended project outcomes offered in *NCHRP Report 785* ***(4),*** safety performance is a prevalent project performance goal evaluated in PBPD. The six PBPD project examples in *NCHRP Report 785* ***(4)*** demonstrate the prevalence of considering predicted safety performance in design alternatives selection. The *Highway Safety Manual's* (HSM) **(6)** role in all *NCHRP Report 785* ***(4)*** project examples reflects the improved and powerful understanding of the relationship between design dimensions and future safety performance.

CDOT's PBPD procedures will evolve as the prevailing PBPD methodologies and relevant technologies advance. The CDOT designer's current role in the process is namely to understand PBPD's potential benefits to their project, PBPD's potential benefits to the overall system and know who to contact for project specific application. The CDOT designer should contact their Region Traffic Representative as early in project lifecycle as possible to begin PBPD coordination.

For example, in a scenario where safety performance is a project performance goal, coordination would be required to enable traffic to provide predicted KAB (fatal crash, incapacitating injury crash, and non-incapacitating injury crash) crashes for the various design alternatives being contemplated within timeframes where the crash forecasts can still factor into design alternative selection. Another application would be to conduct safety performance analysis to understand how a proposed design dimension prompting a design variance request is predicted to relate to future safety performance. In addition to safety performance forecast data, corresponding benefit to cost analysis can be used to evaluate the feasibility of the design variance.

REFERENCES

1. CDOT. *CDOT Online Transportation Information System (OTIS)*

2. AASHTO. *A Policy on Geometric Design of Highways and Streets*, American Association of State Highway and Transportation Officials, Washington, D.C.: 2011.

3. NCHRP. *NCHRP 855, An Expanded Functional Classification System for Highways and Streets*, Transportation Research Board, National Academy Press, Washington D.C., 1993.

4. NCHRP. *NCHRP 785*, *Performance-Based Analysis of Geometric Design of Highways and Streets,* Transportation Research Board, National Academy Press, Washington D.C., 2014

5. FHWA. *Highway Functional Classification Concepts*, *Criteria and Procedures*, *2013 Edition*, Federal Highway Administration, U.S. Department of Transportation, Washington D.C.: 2013. https://www.fhwa.dot.gov/planning/processes/statewide/related/highway_functional_classifications/fcauab.pdf

6. AASHTO. *The Highway Safety Manual*, American Association of State Highway Transportation Professionals, Washington, D.C., 2010

Chapter 2
DESIGN CONTROLS AND CRITERIA

2.0 INTRODUCTION

Chapter 2 of the *PGDHS* **(1)** discusses characteristics of vehicles, pedestrians and traffic that act as criteria for the optimization or improvement in design of the various highway and street functional classes.

2.1 DESIGN VEHICLES

2.1.1 General Characteristics

Selection of design criteria should be determined and documented.

The physical characteristics and the proportions of vehicles of various sizes using the highway are key controls in geometric highway design. Four general classes of design vehicles have been established as follows:

- Passenger cars
  - passenger cars of all sizes, sport/utility vehicles, minivans, vans, and pickup trucks
- Buses
  - intercity (motor coaches), city transit, school, and articulated buses
- Trucks
  - single-unit trucks, truck tractor-semitrailer combinations, and truck tractors with semitrailers in combination with full trailers
- Recreational vehicles
  - motor homes, cars with camper trailers, cars with boat trailers, motor homes with boat trailers, and motor homes pulling cars

The bicycle should also be considered a design vehicle where bicycle use is allowed on a highway.

In the design of any highway facility, the designer should consider the largest design vehicle to normally use that facility or a design vehicle with special characteristics appropriate to a particular intersection in determining the design of such critical features as radii at intersections and radii of turning roadways. In addition, consider the following when selecting a design vehicle:

- A passenger car may be selected when the main traffic generator is a parking lot or series of parking lots.
- A two-axle single-unit truck may be used for intersection design of residential streets and park roads.
- A three-axle single-unit truck may be used for the design of collector streets and other facilities where larger single-unit trucks are likely.

- A city transit bus may be used in the design of state highway intersections with city streets that are designated bus routes and that have relatively few large trucks using them.
- Depending on expected usage, a large school bus (84 passengers) or a conventional school bus (65 passengers) may be used for the design of intersections of highways with low- volume county highways and township/local roads under 400 ADT. The school bus may also be appropriate for the design of some subdivision street intersections.
- See Table 9-3 for the minimum size design vehicle considered for intersections of freeway ramp terminals with arterial crossroads and for other intersections on state highways and industrialized streets that carry high volumes of traffic and/or that provide local access for large trucks.

2.1.2 Minimum Turning Paths of Design Vehicles

The *PGDHS* **(1)** includes drawings and tables to be referenced for minimum turning paths for typical design vehicles.

It is recommended that the Tables 2-2a and 2-2b in the *PGDHS* **(1)** be applied for the appropriate design vehicle. Confirm that the chosen turning radius design will function as planned by using turning template software.

2.2 TRAFFIC CHARACTERISTICS

2.2.1 Volume

Refer to the CDOT website for Traffic Data, including volume, and also the *CDOT Online Transportation Information System (OTIS)* **(2).** Also, see Section 4.01 of the *CDOT Project Development Manual* **(3)** for guidance on traffic data.

2.3 HIGHWAY CAPACITY

2.3.1 Levels of Service

The Region Traffic Engineering Section should be consulted to obtain traffic counts to ascertain if a highway capacity analysis is necessary. Refer to the *Highway Capacity Manual* **(4)** and associated software to determine the effect of design improvements on the level of service.

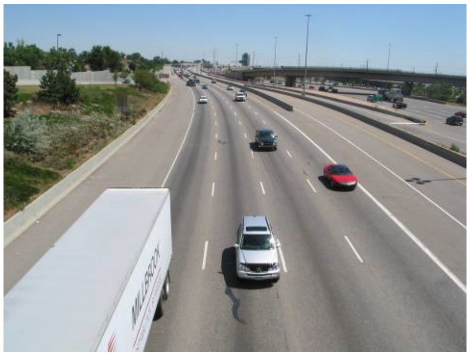
See Table 2-1 for definitions of levels of service and Table 2-2 for design levels of service.

Level of Service	General Operating Conditions
A	Free flow
B	Reasonably free flow
C	Stable Flow
D	Approaching unstable flow
E	Unstable flow
F	Forced or breakdown flow
Note: Specific definitions of level of service A through F vary by facility type and are presented in the Highway Capacity Manual (4).	

Table 2-1 General Definitions of Level of Service

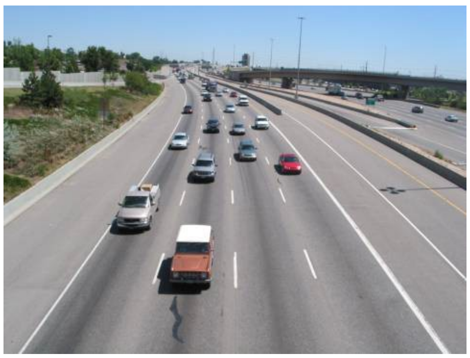
Functional Class	Appropriate Level of Service for Specified Combinations of Area and Terrain Type			
	Rural Level	Rural Rolling	Rural Mountainous	Urban and Suburban
Freeway	B	B	C	C or D
Arterial	B	B	C	C or D
Collector	C	C	D	D
Local	D	D	D	D
Note: While this table provides guidance, engineers should strive to provide the most practical level of service for the conditions/facility.				

Table 2-2 Guidelines for Selection of Design Levels of Service Characteristics by Highway Type



Level of Service A

Level of Service B



Level of Service C

Level of Service D

Level of Service E

Level of Service F

Figure 2-1 Levels of Service

2.4 ACCESS CONTROL AND ACCESS MANAGEMENT

Refer to Chapter 11 and the *State Highway Access Code* **(5)** for further information.

2.5 PEDESTRIANS

Refer to Chapter 12 and Chapter 14.

2.6 SAFETY

Refer to Chapter 20.

2.7 ENVIRONMENT

See Section 3 of the *CDOT Project Development Manual* **(3)** and consult with the Region Environmental Section for information on environmental issues.

REFERENCES

1. AASHTO. *A Policy on Geometric Design of Highways and Streets*, American Association of State Highway and Transportation Officials, Washington, D.C.: 2011.

2. CDOT. *CDOT Online Transportation Information System (OTIS)*: [http://dtdapps.coloradodot.info/Otis]

3. CDOT. *CDOT Project Development Manual*, Colorado Department of Transportation, 2013 (with revisions through 2016).

4. TRB. *Highway Capacity Manual*, Transportation Research Board, Washington, D.C.: 2010.

5. State Highway Access Code, 2 CCR 601-1, as adopted and amended by the Transportation Commission of Colorado, Revised March 2002.

CHAPTER 3
ELEMENTS OF DESIGN

3.0 INTRODUCTION

The design of highways and streets within particular functional classes is treated separately in later chapters. However, common to all classes of highways and streets are several principal elements of design, which include sight distance, superelevation, traveled way widening, grades, and horizontal and vertical alignments. These alignment elements are discussed in this chapter, and, as appropriate, in the later chapters pertaining to specific highway functional classes.

The alignment of a highway or street produces a great impact on the environment, community, and highway user. The alignment is comprised of a variety of elements joined together to create a facility that serves the traffic in a safe and efficient manner, consistent with the facility's intended function. Each alignment element should complement the others to produce a consistent, safe, efficient, and environmentally responsible design.

3.1 SIGHT DISTANCE

3.1.1 General Considerations

A critical element in assuring safe and efficient operation of a vehicle on a highway is the ability to see ahead. Sight distance is the distance along a roadway throughout which an object of specified height is continuously visible to the driver. This distance is dependent on the height of the driver's eye above the road surface; the specified object height above the road surface; and the height and lateral position of sight obstructions such as cut slopes, guardrail, and retaining walls within the driver's line of sight. Sight distance of sufficient length must be provided to allow drivers to avoid striking unexpected objects in the traveled way. Certain two-lane highways should also provide sufficient sight distance to allow drivers to occupy the opposing lane for passing without hazard.

Sight distance falls into three categories:

- Stopping (applicable on all highways)
- Passing (applicable only on two-lane highways)
- Decision (applicable at complex locations)

3.1.2 Stopping Sight Distance

Stopping sight distance is the sum of two distances.

- The distance a vehicle travels from the instant the driver sights an object necessitating a stop to the instant the brakes are applied (brake reaction distance), and
- The distance required to stop the vehicle from the instant brake application begins (braking distance).

Stopping sight distance is measured from the driver's eyes, which are assumed to be 3.5 feet above the pavement, to an object 2 feet high on the road. Distances greater than the minimum stopping sight distance provide an additional measure of safety and should be considered where practical.

Stopping sight distances may be determined directly by calculating braking distance and brake reaction distance, and adding these values together, as described in the *PGDHS* **(1)**. Table 3-1 of this Guide shows sight distances for level roadways and roadways with grade for various design speeds. See also Section 3.1.2.1 for adjustments for grades.

Design Speed	Stopping Sight Distance (Design Values)									Passing Sight Distance (2-Lane Road)	
	No grade adjustment	% Down Grade			% Up Grade			Crest	Sag	Crest Vertical Curve	
(mph)	Dist. (ft)	3	6	9	3	6	9	K	K	Dist. (ft)	K
15	80	80	82	85	75	74	73	3	10	400	57
20	115	116	120	126	109	107	104	7	17		
25	155	158	165	173	147	143	140	12	26	450	72
30	200	205	215	227	200	184	179	19	37	500	89
35	250	257	271	287	237	229	222	29	49	550	108
40	305	315	333	354	289	278	269	44	64	600	129
45	360	378	400	427	344	331	320	61	79	700	175
50	425	446	474	507	405	388	375	84	96	800	229
55	495	520	553	593	469	450	433	114	115	900	289
60	570	598	638	686	538	515	495	151	136	1000	357
65	645	682	728	785	612	584	561	193	157	1100	432
70	730	771	825	891	690	658	631	247	181	1200	514
75	820	866	927	1003	772	736	704	312	206	1300	604
80	910	965	1035	1121	859	817	782	384	231	1400	700
AASHTO Table **(1)**	(3-1)	(3-2)						(3-34)	(3-36)	(3-4) (3-35)	(3-35)

Table 3-1 Sight Distance

$$K = \frac{L}{A} \qquad [3\text{-}1]$$

Where:

L= Length of curve, ft

A= Algebraic difference in intersecting grades, in percent

K value is a coefficient by which the algebraic difference in grade may be multiplied to determine the length in feet of the vertical curve that will provide minimum sight distance. Values of K=167 or greater should be checked for drainage.

3.1.2.1 Effect of Grade on Stopping Sight Distance

The safe stopping distances on upgrades are shorter; those on downgrades are longer. Design speed is used in calculating downgrade corrections; average running speed in calculating upgrade corrections. The different criteria for descending and ascending grades are based on the effect grades have on the speed of individual vehicles, particularly trucks; the effect these vehicles have on the overall speed of the traffic stream; and the premise that many drivers, particularly those in automobiles, do not compensate completely for the changes in speed caused by grades.

On nearly all roads and streets, the grade is traversed by traffic in both directions, but the sight distance at any point on the highway generally is different in each direction, particularly on straight roads in rolling terrain. As a general rule, the sight distance available on downgrades is larger than on upgrades, more or less automatically providing the necessary corrections for grade. Exceptions are one-way roads or streets, as on divided highways with independent design profiles for the two roadways, for which the separate grade corrections are in order and the refinement in design is in keeping with the overall standards used.

For those areas where there is a high volume of trucks, review “Variations for Trucks” in Chapter 3 of the *PGDHS* **(1)**, see Table 3-2 for grade adjustments.

3.1.3 Decision Sight Distance

Table 3-3 of the *PGDHS* **(1)** provides values for appropriate decision sight distances at critical locations and for criteria in evaluating the suitability of the sight lengths at these locations.

Stopping sight distance may not be adequate when drivers are required to make complex or instantaneous decisions, when information is difficult to perceive, or when unexpected or unusual maneuvers are required. In these instances, stopping sight distances may not provide sufficient visibility distance for drivers to corroborate advance warnings and to perform the necessary maneuvers. Decision sight distance provides the greater length that drivers need.

Decision sight distance is the distance required for a driver to:

- Detect an unexpected or otherwise difficult to perceive information source or hazard in a roadway environment that may be visually cluttered.
- Recognize the hazard or its threat potential.
- Select an appropriate speed and path.
- Initiate and complete the required safety maneuver safely and efficiently.

Drivers need decision sight distances whenever there is a likelihood for error in either information reception, decision making, or control actions. The following are examples of critical locations where these kinds of errors are likely to occur and where it is desirable to provide decision sight distance:

- Interchange and intersection locations where unusual or unexpected maneuvers are required.
- Changes in cross section such as toll plazas and lane drops.
- Areas of concentrated demand where there is apt to be "visual noise" whenever sources of information compete, as those from roadway elements, traffic, traffic control devices, and advertising signs.

3.1.4 Sight Distance on Horizontal Curves

Stopping sight distance on horizontal curves may be obtained with the aid of Figures 3-1 and 3-2 and Table 3-1 of this Guide and Figure 3-1 of the *PGDHS* **(1)**. For passenger vehicles, it is assumed that the driver's eyes are 3.5 feet above the center of the inside lane (inside with respect to the curve) and the object is 2 feet high. The line of sight is assumed to intercept the obstruction at the midpoint of the sight line and 2 feet above the center of the inside lane. The middle horizontal sightline offset (HSO) is obtained from Figure 3-1.

Horizontal sight distance may be measured with a straightedge, as indicated in Figure 3-2 of the *PGDHS* **(1).**

As a matter of general case, consider the following:

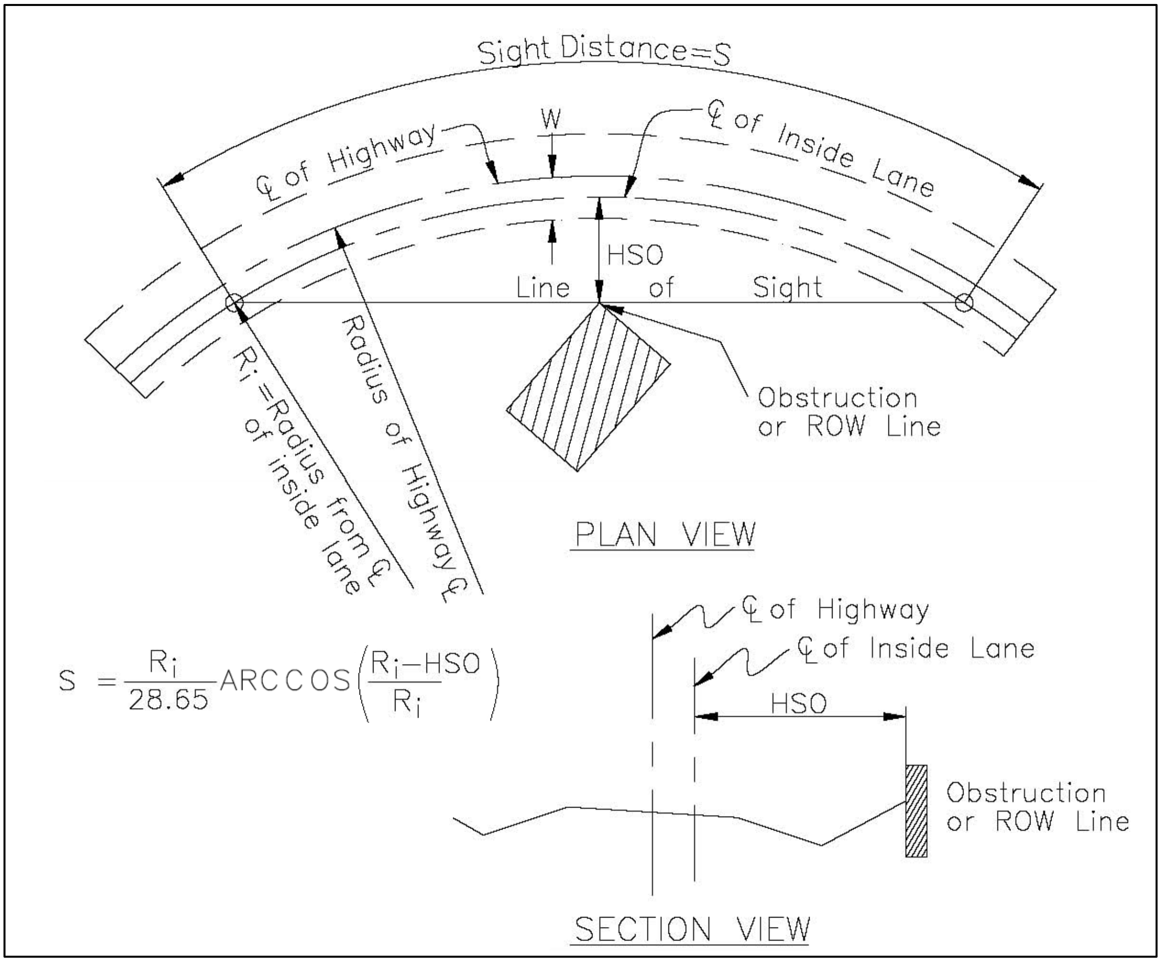


Figure 3-1 General Case – Stopping Sight Distance on Horizontal Curves

Where:

R_i = radius from centerline (C.L.) of inside lane (feet).

HSO = horizontal sightline offset (feet) lateral distance from centerline of inside lane to ROW line or obstruction.

S = available stopping sight distance (feet)

To obtain Stopping Sight Distance, consider the following:

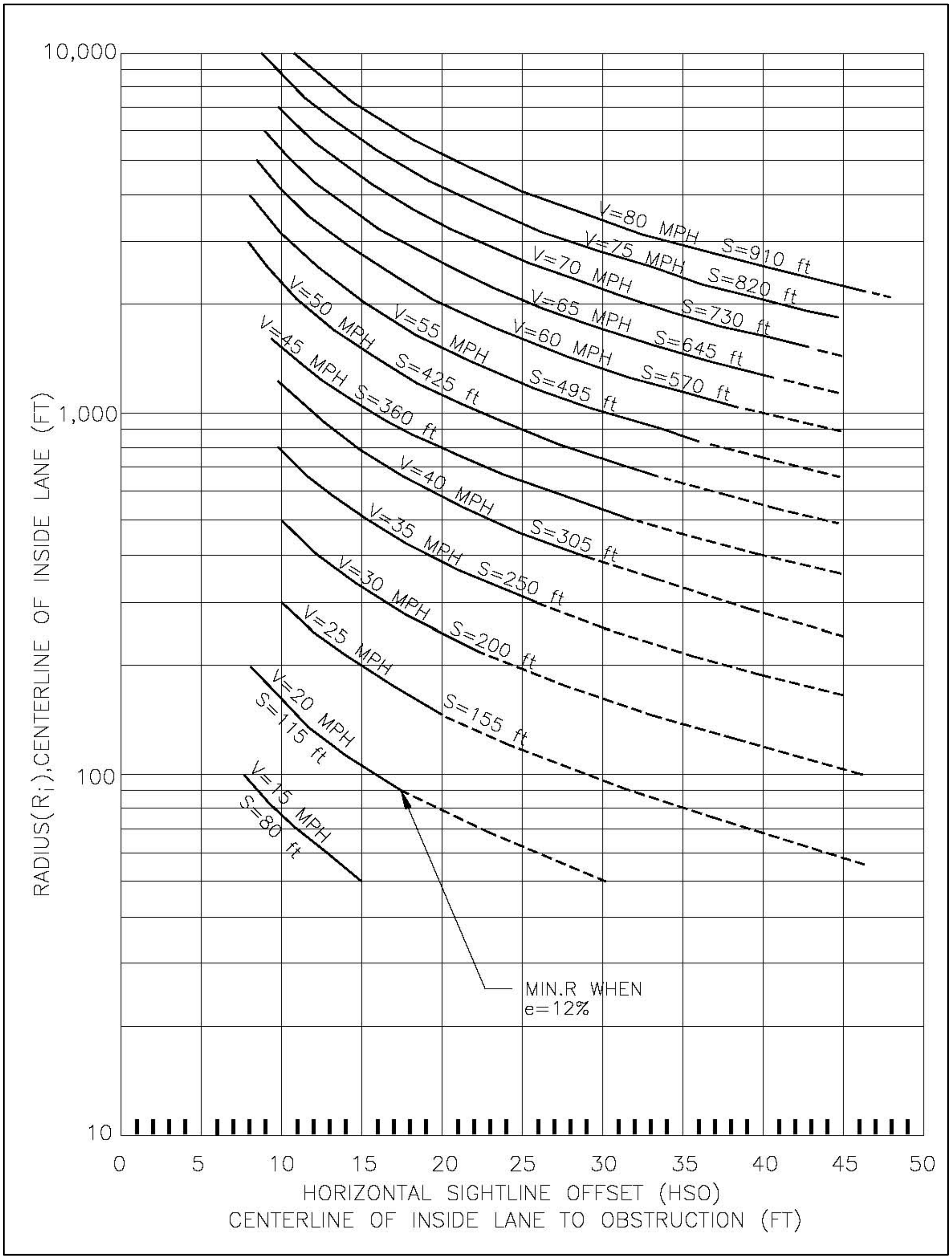


Figure 3-2 [Figure 3-22b of the *PGDHS* (1)] Design Controls for Stopping Sight Distance on Horizontal Curves

Note: this figure does not consider the effects of grade.

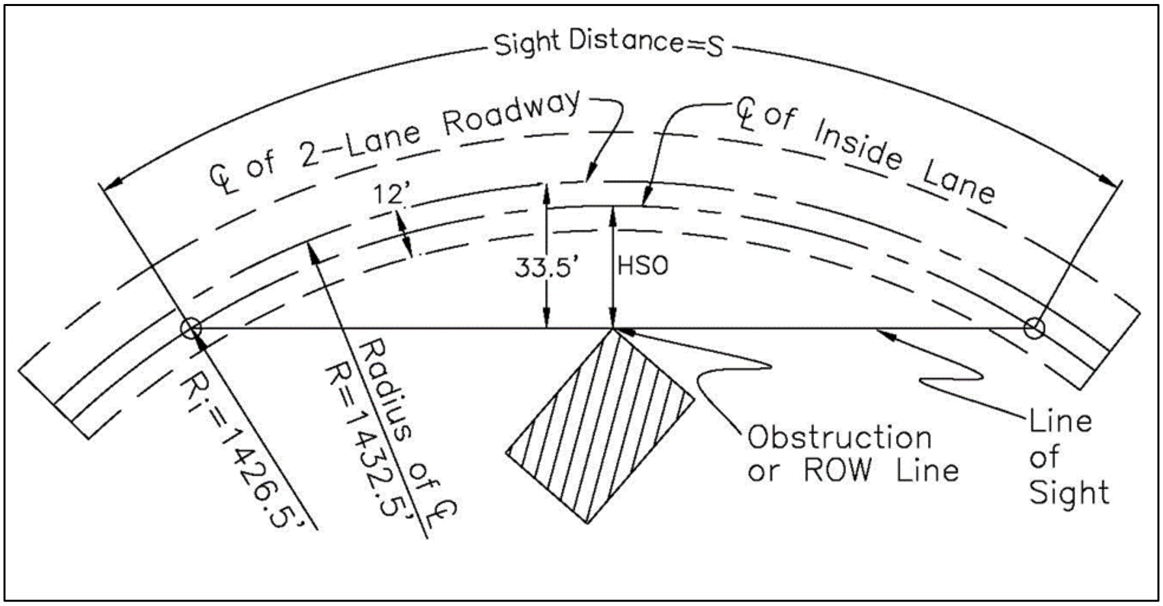


Figure 3-3 Example of Horizontal Stopping Sight Distance on a Two-Lane Roadway

$$R_i = 1432.5 - \frac{12}{2} = 1426.5ft$$

$$HSO = 33.5 - \frac{12}{2} = 27.5ft$$

$$\therefore S = \frac{1426.5}{28.65} \arccos\left(\frac{1426.5 - 27.5}{1426.5}\right) = 561ft$$

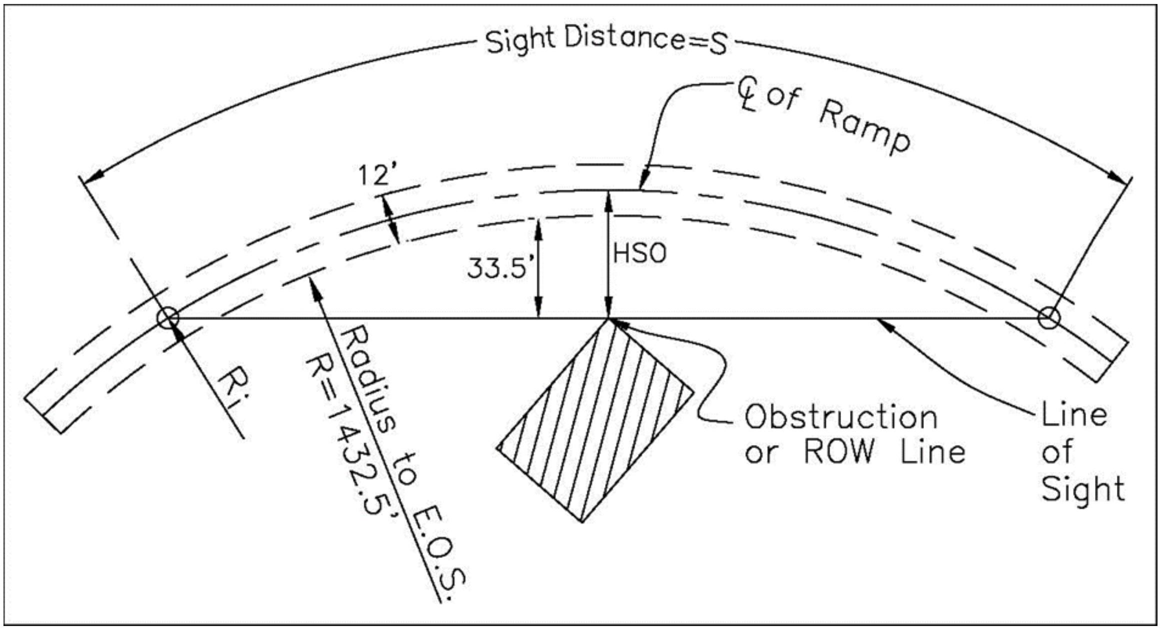


Figure 3-4 Example of Horizontal Stopping Sight Distance on a Ramp

$$R_i = 1432.5 + \frac{12}{2} = 1438.5ft$$

$$HSO = 33.5 + \frac{12}{2} = 39.5ft$$

$$\therefore S = \frac{1438.5}{28.65} \arccos\left(\frac{1438.5 - 39.5}{1438.5}\right) = 676ft$$

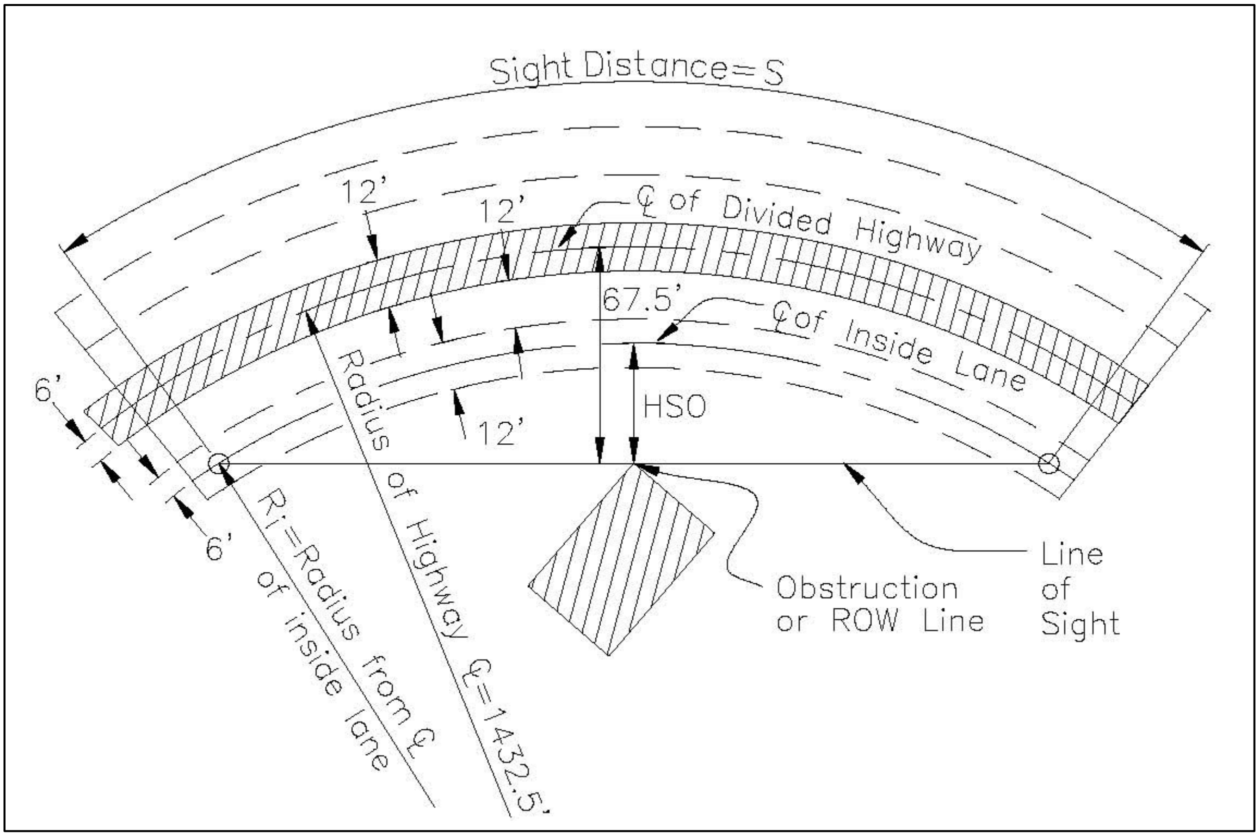


Figure 3-5 Example of Horizontal Stopping Sight Distance on a Divided Highway

$$R_i = 1432.5 - \frac{12}{2} - 12 - \frac{12}{2} = 1408.5ft$$

$$HSO = 67.5 - \frac{12}{2} - 12 - \frac{12}{2} = 43.5ft$$

$$\therefore S = \frac{1408.5}{28.65} \arccos\left(\frac{1408.5 - 43.5}{1408.5}\right) = 702ft$$

Note: If a divided highway has median barrier, the horizontal stopping sight distance for the inside lane of the opposite direction should also be checked and shoulder widening considered.

3.1.5 Sight Distance on Vertical Curves

3.1.5.1 Crest Vertical Curves

Stopping sight distance is measured when the height of eye and the height of object are 3.5 feet and 2 feet respectively.

When S is less than L,

$$S = \sqrt{\frac{2158L}{A}} \qquad [3\text{-}2]$$

When S is greater than L,

$$S = \frac{L}{2} + \frac{1079}{A} \qquad [3\text{-}3]$$

Where:

L = length of vertical curve, in feet
A = algebraic difference in grades, in percent
S = Sight distance, in feet

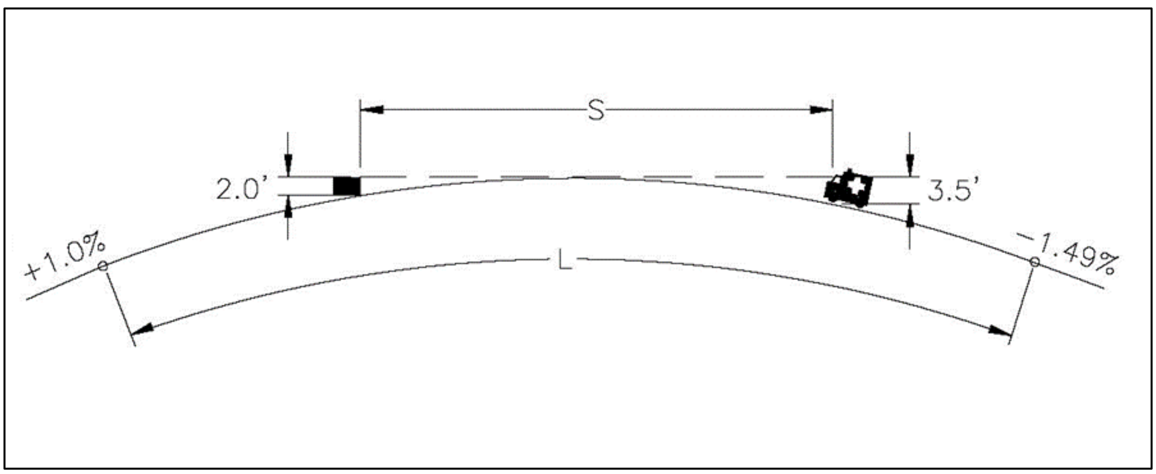


Figure 3-6 Example of Crest Vertical Curve

Given:
L = 400 feet, find S, K and design V.
A = 1.0 – (-1.49) = 2.49

Since it is unknown whether S<L or S>L,

Try(i):

$$S = \sqrt{\frac{2158L}{A}} = \sqrt{\frac{(2158)(400)}{2.49}}$$

S = 589 ft
S(589) > L(400) No Good

Try (ii):

$$S = \frac{L}{2} + \frac{1079}{A} = \frac{400}{2} + \frac{1079}{2.49}$$

$$S = 633ft > L(400) \text{ OK}$$

$$K = \frac{L}{A} = \frac{400}{2.49} = 160.6$$

From Table 3-1 for a K (CREST) value of 160.6, V=60mph

3.1.5.2 Sag Vertical Curves

Headlight sight distance is the basis for determining the length of sight distance. Prior to calculating the following formula, review Figure 3-44 of the *PGDHS* **(1)** to ascertain if S is less than or greater than L.

When S is less than L,

$$S = \frac{3.5L \pm \sqrt{12.25L^2 + 1600AL}}{2A} \quad [3\text{-}4]$$

When S is greater than L,

$$S = \frac{AL + 400}{2A - 3.5} \quad [3\text{-}5]$$

Where:
L = length of sag vertical curve, in feet
S = light beam distance, in feet
A = algebraic difference in grades, in percent

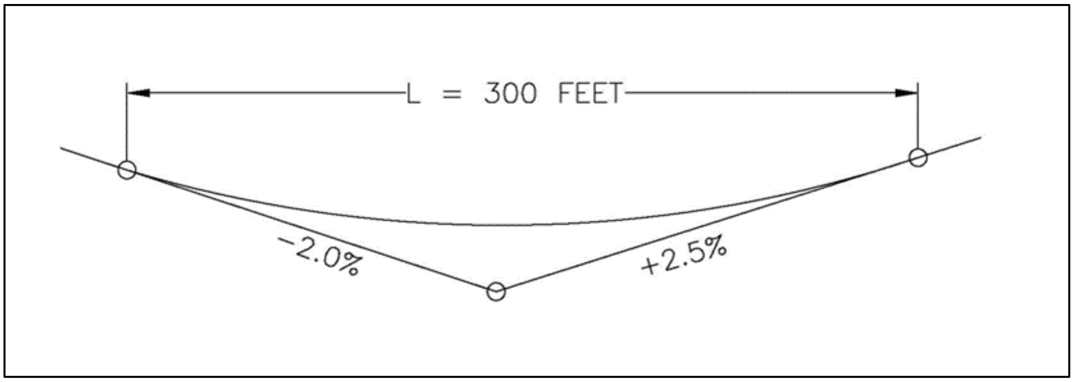


Figure 3-7 Example of Sag Vertical Curve

Given:

L = 300 feet, check if curve is adequate for a design speed of 40 mph, and find S.
A = 2.5 – (-2.0) = 4.5

$$K = \frac{L}{A} = \frac{300}{4.5} = 66.7$$

From "Sag K" column of Table 3-1, with K(Sag) value of 66.7, V = 40.9 mph.
∴ Curve is adequate for a design speed of 40 mph.

Since it is unknown whether S < L or S > L, try each equation or consult Figure 3-44 of the *PGDHS* **(1)**.

Try (iii):

$$S = \frac{3.5L \pm \sqrt{12.25L^2 + 1600AL}}{2A} = \frac{(3.5)(300) \pm \sqrt{(12.25)(300)^2 + (1600)(4.5)(300)}}{(2)(4.5)} =$$

= 317 ft, which is > 300…… No Good!

Try (iv):

$$S = \frac{AL + 400}{2A - 3.5} = \frac{(4.5)(300) + 400}{(2)(4.5) - 3.5} = 318 \text{ ft, which is} > 300 \ldots\text{. OK!}$$

3.1.6 Passing Sight Distance

Passing sight distance is the minimum sight distance required for the driver of one vehicle to pass another vehicle safely. Passing sight distance is considered only on two-lane roads. Passing sight distance is measured between an eye height of 3.5 feet and an object height of 3.5 feet. Table 3-1 presents minimum passing sight distances for various design speeds.

Generally, it is impractical to design crest vertical curves to provide for passing sight distance because of the high cost where crest cuts are involved and the difficulty of fitting the required long vertical curves to the terrain, particularly for high speed roads.

Passing sight distance calculations are for design purposes only to assist in providing as many passing opportunities as possible. Actual passing and no-passing zone locations for striping need to be field measured and placed in accordance with the *Manual on Uniform Traffic Control Devices (MUTCD)* **(2).**

3.1.6.1 Passing Sight Distance on Crest Vertical Curves

Design values of crest vertical curves for passing sight distance differ from those for stopping sight distance because of the different height criterion; i.e., 3.5 feet for the height of object for passing sight distance compared to 2 feet for stopping sight distance. The following formulas apply:

When S is less than L,

$$S = \sqrt{\frac{2800L}{A}} \qquad [3\text{-}6]$$

When S is greater than L,

$$S = \frac{L}{2} + \frac{1400}{A} \qquad [3\text{-}7]$$

For minimum passing sight distances, the required lengths of crest vertical curves are substantially longer than those for stopping sight distances, as evidenced by the values in Table 3-1. These lengths are significantly greater than the lengths necessary for stopping sight distances.

3.1.6.2 Passing and Stopping Sight Distances at Undercrossings

If economically feasible, passing sight distance should be maintained as the highway passes under a structure. On occasion, topographic conditions may result in a pronounced sag curve and the underside of the structure may limit the sight distance. Such conditions may best be checked graphically on the profile using the vertical clear dimension of the structure, the height of the eye for a truck driver as 7.6 feet and the height of object as 2 feet for the taillights of a vehicle.

Minimum stopping sight distance must be maintained. See Table 3-45 of the *PGDHS* **(1)**.

3.2 HORIZONTAL ALIGNMENT

3.2.1 General Controls

Horizontal alignment should provide for safe and continuous operation of vehicles at a uniform design speed for substantial lengths of highway.

The major considerations in horizontal alignment are:

- Topography
- Type of facility
- Design speed
- Profile grade
- Subsurface conditions
- Existing highway and cultural development
- Likely future developments
- Location of the highway terminals
- Right of way
- Safety
- Construction costs
- Environmental issues
- Geological features
- Drainage

All the above considerations should be balanced to produce an alignment that is appropriate for the location and functional classification of the highway. (Functional classification is explained in Chapter 1.)

To a large extent, topography controls both curve radius and design speed. In mountainous areas or areas subject to icing, consideration should be given to locating the road so that a southern exposure will be obtained wherever possible.

Geological features that may affect design, such as potential slide areas and subsurface water, should be investigated by the Materials and Geotechnical Branch.

Sight distance, compatible with the selected design speed, is required for proper design. Stopping sight distances are discussed in 3.1 Sight Distance.

Horizontal alignment must afford at least the minimum stopping sight distance for the design speed at all points on the highway, as given in Table 3-1.

Every effort should be made to exceed the minimum curve radii. Minimum curve radii should be used only when the cost of realizing a higher standard is not consistent with the benefits. The final considerations for the safety of any curve should be the combination of the factors of radius, sight distance, and superelevation (see section 3.2.3).

To avoid the appearance of inconsistent distribution, the horizontal alignment should be coordinated carefully with the profile design. General controls for this combination are discussed in section 3.4.

3.2.2 Types and Properties of Horizontal Alignments

3.2.2.1 Simple Curves

A simple curve is a circular arc joining two tangents.

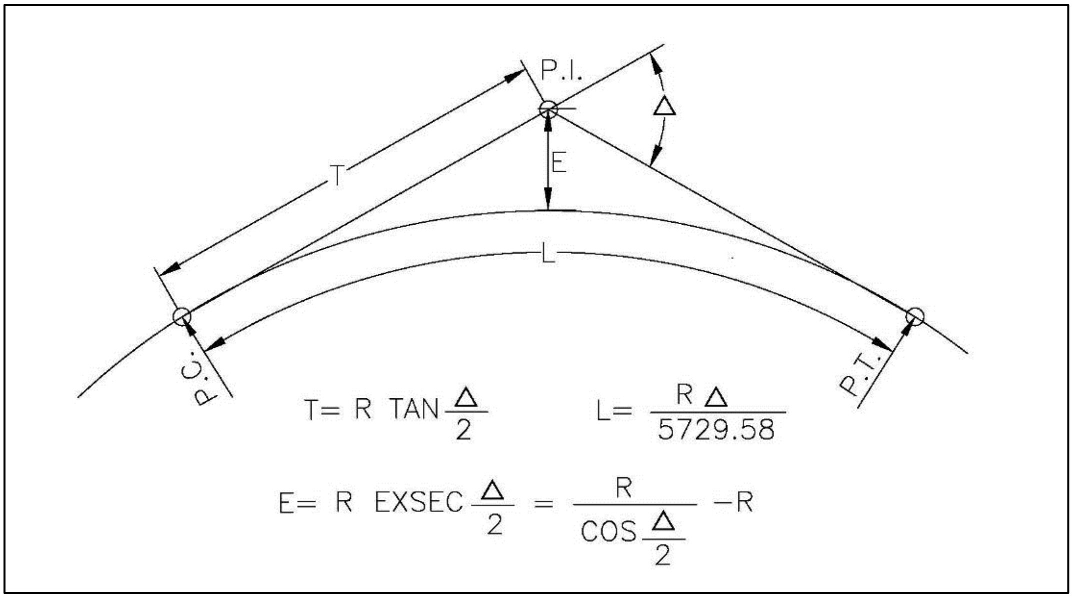


Figure 3-8 Simple Curve

Where:

R = radius of curve, ft
L = length of curve in stations
Δ = deflection angle between the tangents, decimal degrees
T = length of tangent, ft
E = external distance, ft

3.2.2.2 Spiral Curves

Spiral curves provide a gradual change in curvature from a straight to a circular path. Spiral transitions are not required but may be used on all roadways including ramps where recommended by the *CDOT Standard Plans - M & S Standards* **(3)** on superelevation, which also includes minimum transition lengths to be used with any given curvature and speed.

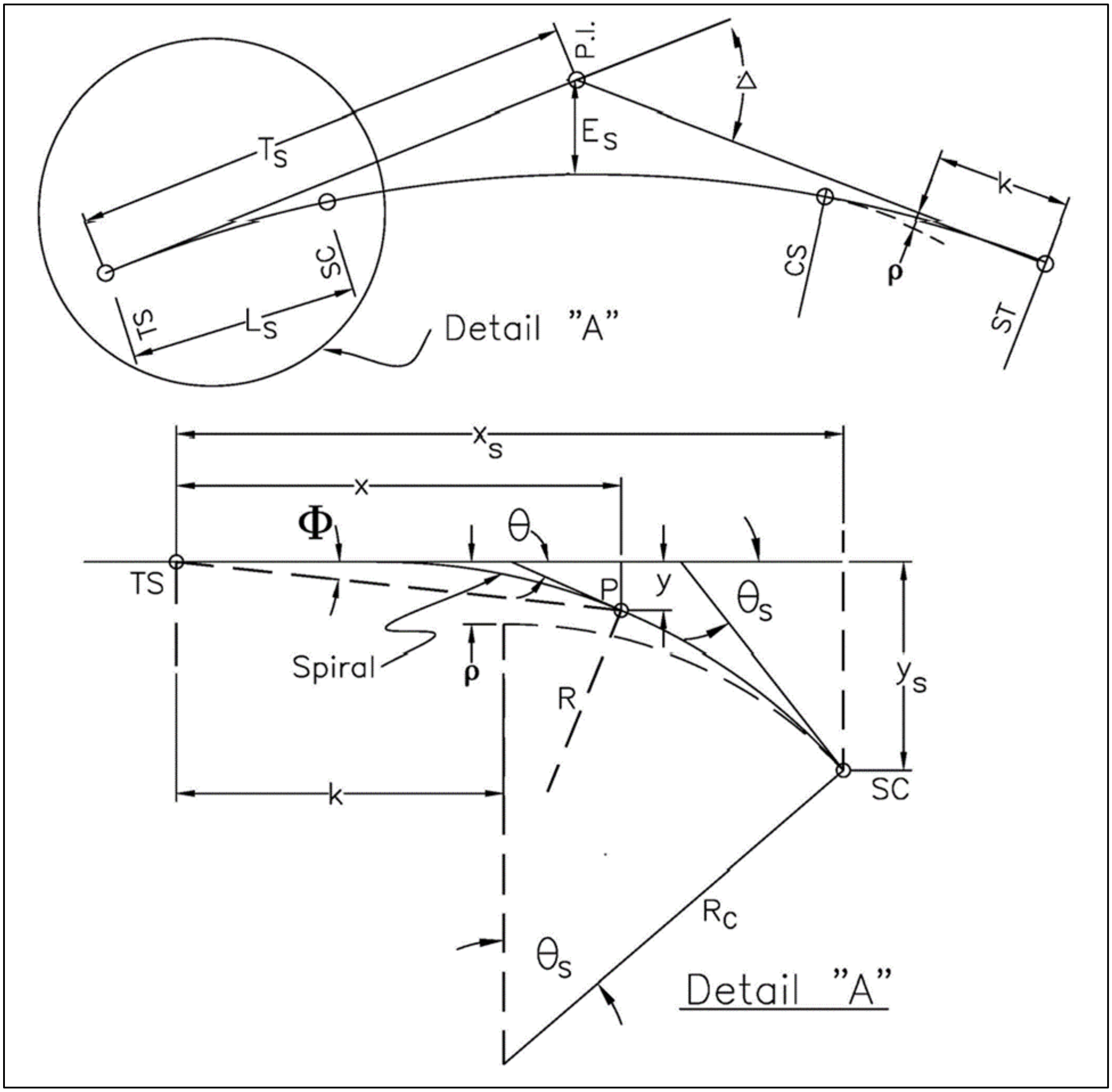


Figure 3-9 Spiral Curve

P is any point on the spiral curve.

Equations for the spiral curve are as follows:

$$T_s = (R_c + \rho)\tan\left(\frac{\Delta}{2}\right) + k \qquad [3\text{-}8]$$

$$E_s = (R_c + \rho)\left(\frac{1}{\cos\left(\frac{\Delta}{2}\right)} - 1\right) + \rho = \frac{R_c + \rho}{\cos\left(\frac{\Delta}{2}\right)} - R_c \qquad [3\text{-}9]$$

$$L_s = \frac{200\ \theta_s}{D_c} \quad [3\text{-}10]$$

$$\theta = \left(\frac{L}{L_s}\right)^2 \theta_s \quad [3\text{-}11]$$

$$D = \left(\frac{L}{L_s}\right) D_c \quad [3\text{-}12]$$

$$y = \frac{L_s}{100}(0.5818\ \theta_s - 0.1266 \times 10^{-4}\ \theta_s^3) \quad [3\text{-}13]$$

$$x = \frac{L_s}{100}(100 - 0.3046 \times 10^{-2}\ \theta_s^3) \quad [3\text{-}14]$$

Where L is in feet and θ is measured in decimal degrees.

$$\rho = 0.001454\ \theta_s\ L_s \quad [3\text{-}15]$$

$$k = L_s(0.5 - 5.0770 \times 10^{-6}\ \theta_s^2) \quad [3\text{-}16]$$

Where θ is measured in decimal degrees.

Where:

TS = point of change from tangent to spiral
SC = point of change from spiral to circle
CS = point of change from circle to spiral
ST = point of change from spiral to tangent
L = spiral arc from the TS to any point on spiral
L_s = total length of spiral from TS to SC
θ = central angle of spiral arc L
θ_s = central angle of spiral arc L_s, called "spiral angle"
Φ = spiral deflection angle at the TS from initial tangent to any point on spiral
D = degree of curve of the spiral at any point
R = radius
D_c = degree of curve of the shifted circle to which the spiral becomes tangent at the SC
R_c = radius of curve of the shifted circle to which the spiral becomes tangent at the SC
Δ = total central angle of the circular curve
Δ_c = central angle of circular arc of length L_c extending from SC to CS
y = tangent offset of any point on spiral with reference to TS and initial tangent
y_s = tangent offset at the SC
x = tangent offset of any point on spiral with reference to TS and initial tangent
x_s = tangent distance for the SC
ρ = offset from the initial tangent to the PC of the shifted circle
k = abscissa of the shifted PC referred to the TS
T_s = total tangent distance = distance from PI to TS, or from PI to ST
E_s = total external distance

For further information on spiral curves, see *Route Location and Design* by T .H. Hickerson **(4)**.

3.2.2.3 Reverse Curves

Two consecutive circular curves constitute a reverse curve if they join at a point of tangency where their centers are on opposite sides of the common tangent. True reversing curves should be avoided, although they may at times be used in designing detours. In cases of reversing curves, a sufficient tangent should be maintained to avoid overlapping of the required superelevation runoff and tangent runout (see section 3.2.3)

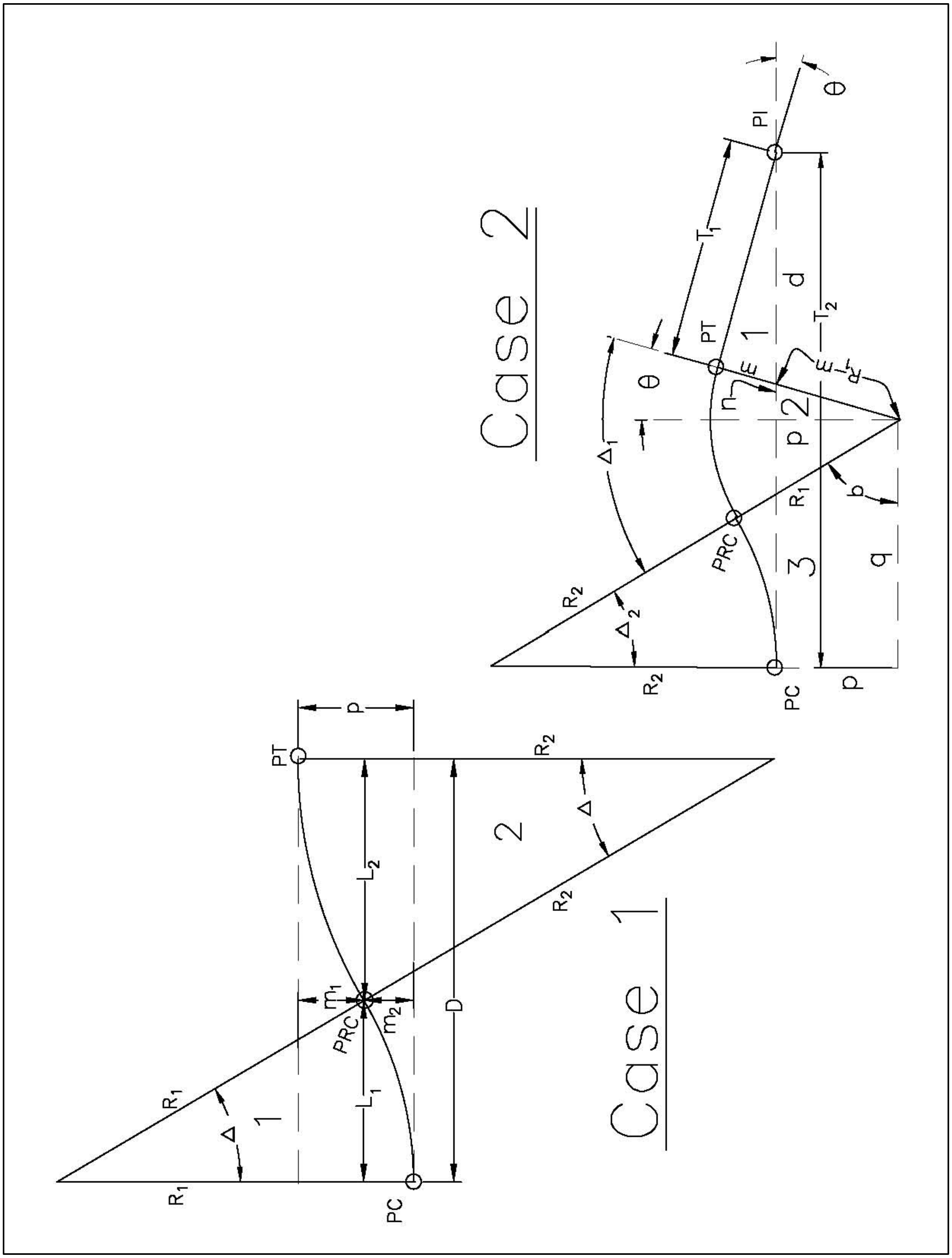


Figure 3-10 Reverse Curves

PRC = Point of Reversing Curvature.

In case 1, the two parallel tangents are to be connected by a reversed curve (such as in a detour). R_1, R_2, and p are given.

From triangle 1,

$$\frac{L_1}{R_1} = \sin\Delta$$

$$L_1 = R_1 \sin\Delta$$

And,

$$\frac{R_1 - m_2}{R_1} = \cos\Delta$$

$$m_2 = R_1(1 - \cos\Delta)$$

From triangle 2,

$$\frac{L_2}{R_2} = \sin\Delta$$

$$\frac{R_2 - m_1}{R_2} = \cos\Delta$$

From which,

$$p = m_2 + m_1$$

$$p = R_1(1 - \cos\Delta) + R_2(1 - \cos\Delta)$$

$$p = (R_1 + R_2)(1 - \cos\Delta)$$

$$\frac{p}{R_1 + R_2} = 1 - \cos\Delta \qquad [3\text{-}17]$$

And,

$$D = L_1 + L_2 \qquad [3\text{-}18]$$

$$D = R_1 \sin\Delta + R_2 \sin\Delta$$

$$D = (R_1 + R_2)\sin\Delta \qquad [3\text{-}19]$$

From equations [3-17] [3-18] and [3-19] and the ordinary functions of a simple curve, all ordinary cases of reversed curves between parallel tangents can be solved.

In Case 2, the two tangents, intersecting with the angle θ, are to be connected by the reversed curve in which T_1, R_1, and R_2 are known, and the tangent distance T_2 and the central angles of the two branches (Δ_1 & Δ_2) are required.

In triangle 1, the base T_1 and the angles are known, from which the sides d and m can be computed.

In triangle 2, the hypotenuse is R_l - m, and the angles are known from which the base p and the altitude n are determined.

In triangle 3, the base is R_2 + p, and the hypotenuse is $R_l + R_2$, from which the angles Δ_2 and b and the distance q can be found.

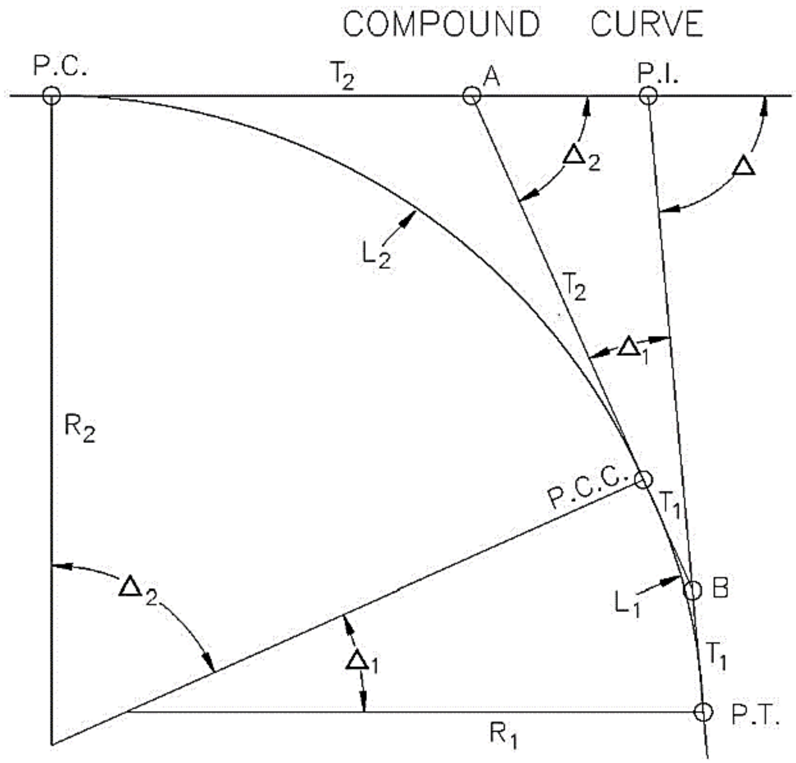
Then
$$\Delta_1 = \theta + \Delta_2 \qquad [3\text{-}20]$$

and
$$T_2 = d + n + q \qquad [3\text{-}21]$$

3.2.2.4 Compound Curves

Compound circular curves are two or more consecutive circular curves in the same direction with varying radii. Compound circular curves are joined at a point of tangency and located on the same side of the common tangent.

While simple curves are preferred, compound curves can be used to satisfy topographical constraints that cannot be as effectively balanced with simple curves. For compound curves on open highways, it is generally accepted that the ratio of the flatter radius to the sharper radius should not exceed 1.5:1. For compound curves at intersections or on ramps, the ratio of the flatter radius to the sharper radius should not exceed 2:1. When this is not feasible, an intermediate simple curve or spiral should be used to provide the necessary transitions. Refer to the *PGDHS* **(1),** Chapter 3 for more discussion on compound curves at intersections.



Given: $\Delta = \Delta_1 + \Delta_2$ $\quad T_2 = R_2 \text{ TAN} \frac{\Delta_2}{2}$ $\quad T_1 = R_1 \text{ TAN} \frac{\Delta_1}{2}$

Curve is laid out as two adjacent simple curves:
one having a radius "R2" with its P.I. at "A";
the other having a radius "R1" with its P.I. at "B".

On open highways, the ratio R2/R1 should not exceed 1.5:1. On other curves (intersections and ramps), the ratio R2/R1 should not exceed 2:1. For sharper curves, use a three-centered curve or a spiral.

Figure 3-11 Compound Curve

3.2.2.5 Alignment on Bridges

Ending a curve on a bridge is undesirable and adds to the complication of design and construction. Likewise, curves beginning or ending near a bridge should be placed so that no part of the spiral or superelevation transitions extends onto the bridge. Compound curves on a bridge are equally undesirable. If curvature is unavoidable, every effort should be made to keep the bridge within the limits of the simple curve.

3.2.2.6 Curvature Zoning

In addition to the specific design elements for horizontal alignment discussed under previous headings, a number of general controls are recognized in practice. These controls are not subject to theoretical derivation, but they are important for efficient and smooth-flowing highways. Excessive curvature or poor combinations of curvature limit capacity, cause economic losses because of increased travel time and operating costs, and detract from a pleasing appearance. To avoid such poor design practices, the general controls that follow should be used where practical.

Consistent alignment should always be sought. Sharp curves should not be introduced at the ends of long tangents. Sudden changes from areas of flat curvature to areas of sharp curvature should be avoided. Where sharp curvature is introduced, it should be approached, where practical, by a series of successively sharper curves.

- **Broken-Back Curve**

  A broken-back curve consists of two curves in the same direction joined by a short tangent (under 1,500 feet). Broken-back curves are undesirable and can typically be replaced by one simple curve. If used, a simple curve, a compound curve or spiral transitions should be used to provide some degree of continuous superelevation. Lengths need to be adequate to transition superelevation correctly.

  The “broken-back” arrangement of curves should be avoided except where very unusual topographical or right of way conditions make other alternatives impractical.

- **Small Deflection Angles**

  For small deflection angles, curves should be sufficiently long to avoid the appearance of a kink. Curves should be at least 500 feet long for a central angle of 5 degrees, and the minimum length should be increased 100 feet for each 1 degree decrease in the central angle. Horizontal curves should not be used when the central angle is 59 minutes or less on non-freeways. The minimum length for horizontal curves on main highways should be fifteen times the design speed (15V). On high-speed controlled facilities that use flat curves for aesthetic reasons, the desired minim length for curves should be thirty times the design speed (30V).

- **Passing Tangents**

  Passing tangents are used to provide passing opportunities on two-lane roads. One-half mile is considered an adequate length. Passing tangents should be provided as frequently as possible in keeping with the terrain.

  An effort to introduce a passing tangent or to increase the length of a passing tangent is always worthy of reasonable expenditure. Nothing is gained if sweeping curves of large radii are used at the ends of a tangent if they reduce its length to less than that required for safe passing. It is better to use somewhat shorter radii and increase the intervening tangent to a more satisfactory length. At the other extreme, sharp curves at the ends of a passing tangent should be avoided as indicated above.

3.2.3 Superelevation

3.2.3.1 General

One of the most important factors to consider in highway safety is the centrifugal force generated when a vehicle traverses a curve. Centrifugal force increases as the velocity of the vehicle and/or the degree of curvature increases.

To overcome the effects of centrifugal force, curves must be superelevated. It is impossible to balance centrifugal force by superelevation alone, because for any given curve radius a certain superelevation rate is exactly correct for only one driving speed. At all other speeds there will be a side thrust either outward or inward, relative to the curve center, which must be offset by side friction. See the *PGDHS* **(1)** for further discussions on side friction.

See section 3.5 for superelevation of detours.

3.2.3.2 Standards for Superelevation

The *CDOT Standard Plans – M & S Standards* **(3)** on Superelevation give the required rate of superelevation for the various radius lengths at different design speeds for the maximum superelevation rate. See *CDOT Standard Plans – M & S Standards* M-203-11, and M-203-12 **(3).** The values in the standard plans match those in Tables 3-8, 3-9, and 3-10 of the *PGDHS* **(1)**.

In general, the highest superelevation rate used on highways in climates with snow and ice should be 8 percent. In practice, the maximum superelevation rate chosen on Colorado highways is typically either 6 or 8 percent after the designer considers the four factors discussed in section 3.3.3 (page 3-30) of the *PGDHS* **(1)**. While a maximum superelevation rate of 8 percent is generally practical elsewhere, a maximum superelevation rate of 6 percent is typically chosen in urban areas. The selection of 6 percent as the maximum superelevation rate is also common on viaducts where freezing and thawing conditions are likely, because bridge decks generally freeze more rapidly than other roadway sections. Where freeways are intermittently elevated on viaducts, the lower superelevation rate should be used throughout for design consistency.

The maximum superelevation rate may be less than shown on *CDOT Standard Plans – M & S Standards* **(3)** when the designer determines that the lower rate is required because of traffic congestion or extensive marginal development that acts to restrict top speeds.

For divided highways where median width is less than 60 feet, future inside widening of bridges or providing additional lanes requires the designer to properly plan the superelevation. Things to consider are:

- superelevation pivot point
- vertical clearance
- superelevation transitions

3.2.3.3 Superelevation Transition

Superelevation runoff is the term denoting the length of highway needed to accomplish the change in cross slope from a section with the adverse crown removed (one side superelevates at normal crown slope, the other side at zero slope) to the fully superelevated section, or vice versa. When a spiral is used, its length is used to accommodate the superelevation runoff.

Tangent runout is the term denoting the length of highway needed to accomplish the change in cross slope from a normal crown section to a section with the adverse crown removed (one side superelevates at normal crown slope, the other side at zero slope), or vice versa.

The length of the tangent runout is determined by the amount of adverse crown to be removed and the rate at which it is removed. This rate of removal should be the same as the rate used to effect the superelevation runoff.

The location, with respect to the curve, and the various lengths of the superelevation transitions are shown on the *CDOT Standard Plans - M & S Standards* **(3)** on Superelevation.

3.2.3.4 Design for All Rural Highways, Urban Freeways and High-Speed Urban Streets

On all rural highways, urban freeways, and urban streets where speed is relatively high and relatively uniform, horizontal curves are generally superelevated and successive curves are generally balanced to provide a smooth-riding transition from one curve to the next.

Refer to section 3.2.3.2, for the recommended maximum superelevation rates for all rural highways, urban freeways, and high-speed urban streets.

Table 3-7 of the *PGDHS* **(1)** gives minimum curve radius in feet for specific design speeds and the rates of superelevation. The table is based on design speed and superelevation alone and does not consider the sight distance factor.

3.2.3.5 Design for Low-Speed Urban Streets

Although superelevation is advantageous for traffic operations, various factors often combine to make its use impractical in many built-up areas. Such factors include wide pavement areas; need to meet the grade of adjacent property; surface drainage considerations; and frequency of cross streets, alleys and driveways. Therefore, horizontal curves on low-speed streets in urban areas are frequently designed without superelevation, counteracting the centrifugal force solely with side friction. On these curves, traffic entering a curve to the left has an adverse or negative superelevation due to the normal crown, but with flat curves and lower speeds the resultant friction required to counteract both the centrifugal force and the negative superelevation is small.

On successively sharper curves for the same design speed, the maximum degree of curvature or sharpest curve without superelevation is reached when the side friction factor developed to counteract centrifugal force and adverse crown reaches the maximum allowable value based on safety and comfort considerations. For travel on sharper curves, superelevation is needed.

The maximum superelevation rate of zero in Table 3-2 establishes the minimum radius for each speed below which superelevation is not provided on local streets in residential and commercial areas but should be considered in industrial areas or other streets where operating speeds will be higher. A maximum superelevation rate of 4 percent or 6 percent is commonly used. The maximum curvature for a given design speed is defined for low-speed urban streets when both the maximum superelevation rate and the maximum allowable side friction factors are utilized.

e	**R (ft) for Design Speed (mph)**						
(%)	**15**	**20**	**25**	**30**	**35**	**40**	**45**
-2.0	50	107	198	333	510	762	1039
-1.5	49	105	194	324	495	736	1000
0	47	99	181	300	454	667	900
1.5	45	94	170	279	419	610	818
2.0	44	92	167	273	408	593	794
2.2	44	91	165	270	404	586	785
2.4	44	91	164	268	400	580	776
2.6	43	90	163	265	396	573	767
2.8	43	89	161	263	393	567	758
3.0	43	89	160	261	389	561	750
3.2	43	88	159	259	385	556	742
3.4	42	88	158	256	382	550	734
3.6	42	87	157	254	378	544	726
3.8	42	87	155	252	375	539	718
4.0	42	86	154	250	371	533	711
4.2	41	85	153	248	368	528	703
4.4	41	85	152	246	365	523	696
4.6	41	84	151	244	361	518	689
4.8	41	84	150	242	358	513	682
5.0	41	83	149	240	355	508	675
5.2	40	83	148	238	352	503	668
5.4	40	82	147	236	349	498	662
5.6	40	82	146	234	346	494	655
5.8	40	81	145	233	343	489	649
6.0	39	81	144	231	340	485	643

Notes:
1. Computed using Superelevation Distribution Method 2.
2. Superelevation may be optional on low-speed urban streets.

Table 3-2 [Table 3-13b of the *PGDHS* (1)] Minimum Radii and Superelevation for Low-Speed Urban Streets

3.2.4 Widths for Turning Roadways at Intersections

See section 9.5.1.

3.2.5 Traveled Way Widening on Horizontal Curves

Curve widening is used primarily on pavements of substandard width or curvature. On open highway curves, the pavement should be widened as shown in Table 3-26b of the *PGDHS* **(1)**, which is based on a WB-62 design vehicle. See Table 3-27 of the *PGDHS* **(1)** for curve-widening information for other design vehicles.

Widening is costly and little is gained from a small amount of widening. A widening of less than 2 feet may be disregarded.

3.2.5.1 Attainment of Widening on Curves

Widening should be attained gradually on the approaches to the curves, as shown in Figure 3-12 and Figure 3-13, to ensure a reasonably smooth alignment of the edge of the pavement and to fit the paths of vehicles entering or leaving the curves.

Widen (W) on the inside edge of the pavement and extend the transition over the same transition length (L) as the superelevation runoff.

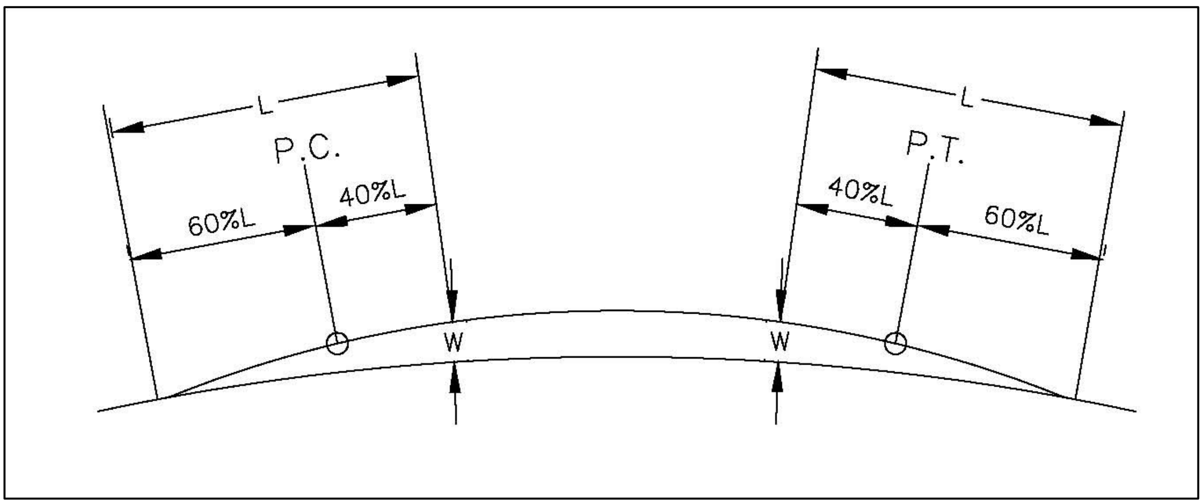


Figure 3-12 Widening on a Simple Curve

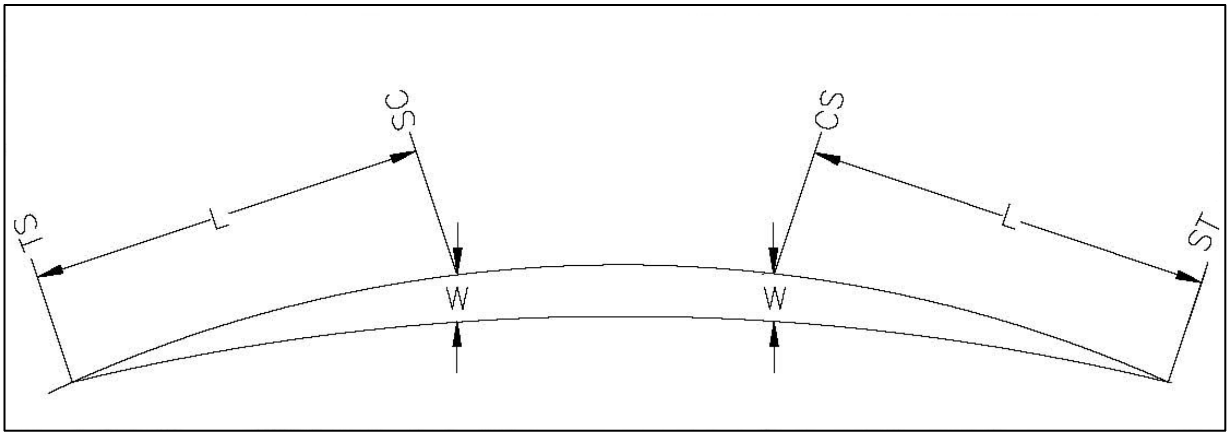


Figure 3-13 Widening on a Spiral Curve

From Section 6C.08 of the *MUTCD* **(2)**.

For V ≤ 40 mph $$L \geq \frac{WV^2}{60}$$ [3-22]

For V > 40 mph $$L \geq WV$$ [3-23]

Example:

Given:

Pavement = 22 feet
Degree of curve = 9°
Radius = 636.62 feet
Icing conditions frequently exist, crowned highway

When V = 30 mph
From Table 3-26b of the *PGDHS* **(1)**, Widening (W) = approximately 3.5 feet
From CDOT *Standard Plans – M & S Standards* **(3)**, Superelevation
Crowned Highways, L = 100 ft

Since V ≤ 40, $\dfrac{WV^2}{60} = \dfrac{3.5(30)^2}{60} = 52.5\ ft$

and $L > \dfrac{WV^2}{60}\ \ OK$

When V = 50 mph and curve radius = 760 feet with other parameters same as above,
From Table 3-26b of the *PGDHS* **(1)**, Widening (W) = approximately 4.3 feet
From CDOT *Standard Plans – M & S Standards* **(3)**, Superelevation
Crowned Highways, L = 240 ft
Since V > 40 mph, WV = 4.3(50) = 215 ft
And L > WV …. OK

3.2.6 Pavement Transitions

3.2.6.1 General

A pavement transition is the area of variable pavement width encountered when changing from one roadway width, or section, to another.

3.2.6.2 Two Lanes to Multilane Divided

This type of transition should be made only where sight distance is not restricted such as on a tangent section or on a flat curve. On a tangent section, the transition may be accomplished on either one or both lanes. A maximum of 1 degree reversing curves and a minimum total transition length of 1,000 feet should be used. This minimum length shall also apply where the transition is accomplished on a curve.

Design standards of the two lanes should be consistent with those of the multilane facility.

3.2.6.3 Other Transitions

Other, more simplified, transitions occur at speed-change lanes (see Chapter 9), truck climbing lanes (see section 3.3.5), and widening for curves (see section 3.2.5.1). All transitions shall be consistent with the design speed for the facility.

3.3 VERTICAL ALIGNMENT

3.3.1 General Controls

The grade line is a reference line by which the elevation of the pavement and other features of the highway are established. It is controlled mainly by topography and structure clearances, but the factors of horizontal alignment, safety, sight distance, design speed, construction costs, and the performance of heavy vehicles on a grade also must be considered.

In flat terrain, the elevation of the grade line is often controlled by drainage considerations.

In rolling terrain, an undulating grade line is often desirable, both from a standpoint of construction and maintenance economy. However, undulating grade lines involving substantial lengths of momentum grades should be appraised for their effect upon traffic operations since they may result in undesirably high downgrade truck speeds.

In mountainous terrain, the grade line is usually closely dependent upon physical controls, although adverse grades should be avoided. On divided highways, independent profiles with grade differential should be considered. Broken-back grade lines should always be avoided.

On long grades, it is preferable to flatten the grades near the top of the ascent particularly on low design speed highways.

In all cases, the consideration of adequate sight distance requirements and other safety factors should take precedence over construction and maintenance costs.

3.3.2 Position With Respect to Cross Section

The grade line should generally coincide with the axis of rotation for superelevation.

- On undivided highways, the grade line should coincide with the highway centerline.
- On ramps and interstate-to-interstate connections, the grade line is generally positioned at the left edge of the traveled way. Either edge of traveled way or centerline may be used on multilane facilities.
- On divided highways, the grade line should be positioned at the centerline of the median for paved medians 60 feet wide or less.

In selecting where the grade line is in relation to the axis of rotation for superelevation, the designer should consider the following:

- Future widening.
- Mountainous terrain.
- Right of way constraints.
- Topographic features.
- Earthwork.
- Matching existing typical sections (as-constructed plans).

Table 3-3 shows clearances to structures and obstructions for the various functional classifications.

<table>
<tr><th rowspan="2"></th><th colspan="2">HIGHWAY UNDERPASSES</th><th colspan="2">RAILWAY UNDERPASSES ***</th><th colspan="2">OVERHEAD WIRES</th></tr>
<tr><th>HORIZONTAL</th><th>VERTICAL*</th><th>HORIZONTAL</th><th>VERTICAL* ‡</th><th>HORIZONTAL▲</th><th>VERTICAL*</th></tr>
<tr><td>Local Rural Roads</td><td rowspan="7">E</td><td rowspan="4">14.5 feet</td><td rowspan="7">F</td><td rowspan="7">G</td><td>A</td><td rowspan="7">H</td></tr>
<tr><td>Local Urban Streets**</td><td>B</td></tr>
<tr><td>Rural Collectors</td><td>C</td></tr>
<tr><td>Urban Collectors**</td><td rowspan="4">D</td></tr>
<tr><td>Rural Arterials</td><td rowspan="3">16.5 feet #</td></tr>
<tr><td>Urban Arterials**</td></tr>
<tr><td>Freeways</td></tr>
</table>

A 10 feet from edge of traveled way.

B Use A when practical, but in any event, provide a minimum of 2 feet from curb face or from shoulder edge. See *AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals* **(5)**.

C ≤40mph use **A**, ≥45mph use **D**

D Use a clear zone according to the *AASHTO Roadside Design Guide* **(6)** or provide guard rail.

E Carry full approach roadway section through structure; minimum clearance from edge of through traffic lanes to walls, piers, or toes of slopes shall correspond to D, but, desirably, should not be less than 30 feet, unless guard rail is used. For tunnels and depressed roadways see the *AASHTO Standard Specifications for Highway Bridges* **(7)**.

F See *CDOT Bridge Design Manual* **(8),** subsection 2.5.3.

G Minimum requirements vary by railroad. BNSF requires 23.5 feet, UPRR requires 23.33 feet (2016 Guidelines for Railroad Grade Separation Projects). PUC and federal minimum requirement is 23 feet per AREMA.

H Communication lines and power lines in accordance with Table 1 of the *State Utility Accommodation Code* **(14)**:

Type of Conductor, Cable & Voltage	**Over Roadway Template (feet)**	**Outside Roadway Template (feet)**
Noninsulated communication conductors; supply cables 0-750 Volts (multiplex wire)	24	20.5
Open Supply Conductors 0 - 750 Volts	24	21
Open Supply Conductors >750 Volts to 22 kVolts	25	23
Voltages exceeding 22 kVolts to 50 kVolts	25◆	23◆
Voltages exceeding 50 kVolts	25.5◆◆	23◆◆
◆plus 0.4 inch per 1,000 Volts in excess of 22 kVolts		
◆◆plus [0.4 inch per 1,000 Volts in excess of 22 kVolts] X [1.0 + (.03 per 1,000 feet above 3,300 feet above sea level)] or alternate method for voltages exceeding 98 kVolts		
Voltages are phase to ground for effectively grounded circuits and those other circuits where all ground faults are cleared by promptly de-energizing the faulted section, both initially and following subsequent breaker operations.		

*All vertical clearances shown make allowance for future overlays or additional ballast on railroad. Vertical clearance applies to the full pavement width, including provisions for future widening.
**For low-speed urban conditions (≤40 mph) see Chapter 13.
***All railway clearances are subject to the individual railroad approval.
#May be reduced to 14.5 feet in Special Cases [See the *PGDHS* **(1)** and *CDOT Bridge Design Manual* **(8),** subsection 2.2.2**]**.
‡Vertical clearance to sign trusses and pedestrian overpasses shall be 17.5 feet.
▲Horizontal clearance is distance to utility poles as well as light poles, fire hydrants, sign poles, and other similar obstructions.

Table 3-3 Clearances to Structures and Obstructions

3.3.3 Standards for Grades

- Minimum grades

  Flat and level grades on uncurbed pavements are acceptable when the pavement is adequately crowned to drain the surface laterally (see Chapter 4).

  With curbed pavements, longitudinal grades should be sufficient to facilitate curb drainage. A minimum curb flowline grade for the usual case is 0.5 percent, but a grade of 0.30 percent may be used where there is a high type pavement adequately crowned and supported on firm subgrade. With curbed sections on sag vertical curves, a grade of at least 0.30 percent should be retained at the curb and gutter line by increasing the crown slope or, if necessary, shortening the vertical curve length to keep the crown slope from exceeding the maximum value given in Chapter 4.

- Maximum grades

  The desirable maximum grades for the various functional classifications are shown in Table 3-4. The maximum design grade should be used infrequently; in most cases, grades should be less than the maximum design grade.

  The term "critical length of grade" is used to indicate the maximum length of a designated upgrade upon which a loaded truck can operate without an unreasonable reduction in speed. On grades longer than "critical," consideration of extra lanes should be made (see section 3.3.5).

Type of Terrain	Maximum Grade (%) for Design Speed (mph)										
	20	30	40	45	50	55	60	65	70	75	80
	RURAL AND URBAN FREEWAYS [a] [Table 8-1 **(1)**]										
LEVEL					4 [c]	4 [c]	3 [c]	3	3	3	3
ROLLING					5 [c]	5 [c]	4 [c]	4	4	4	4
MOUNTAINOUS					6	6	6	5	5	--	--
	RURAL ARTERIALS [Table 7-2 **(1)**]										
LEVEL			5 [c]	5 [c]	4 [c]	4 [c]	3	3	3	3	3
ROLLING			6 [c]	6 [c]	5	5	4	4	4	4	4
MOUNTAINOUS			8	7	7	6	6	5	5	5	5
	URBAN ARTERIALS [Table 7-4 **(1)**]										
LEVEL		8	7	6	6	5	5				
ROLLING		9	8	7	7	6	6				
MOUNTAINOUS		11	10	9	9	8	8				
	RURAL COLLECTORS [b] [Table 6-2 **(1)**]										
LEVEL	7 [c]	7 [c]	7	7	6	6	5				
ROLLING	10 [c]	9	8	8	7	7	6				
MOUNTAINOUS	12	10	10	10	9	9	8				
	URBAN COLLECTORS [b] [Table 6-8 **(1)**]										
LEVEL	9 [c]	9	9	8	7	7	6				
ROLLING	12 [c]	11	10	9	8	8	7				
MOUNTAINOUS	14 [c]	12	12	11	10	10	9				
	LOCAL RURAL ROADS [Table 5-2 **(1)**]										
LEVEL	8 [d]	7	7	7	6	6	5				
ROLLING	11	10	10	9	8	7	6				
MOUNTAINOUS	16	14	13	12	10	10	--				

[a] Grades one percent steeper than the value shown may be used in urban areas.

[b] Maximum grades shown for rural and urban collector conditions of short lengths (less than 500 feet) and on one-way down grades may be two percent steeper.

[c] Design speed shown not recommended (less than minimum).

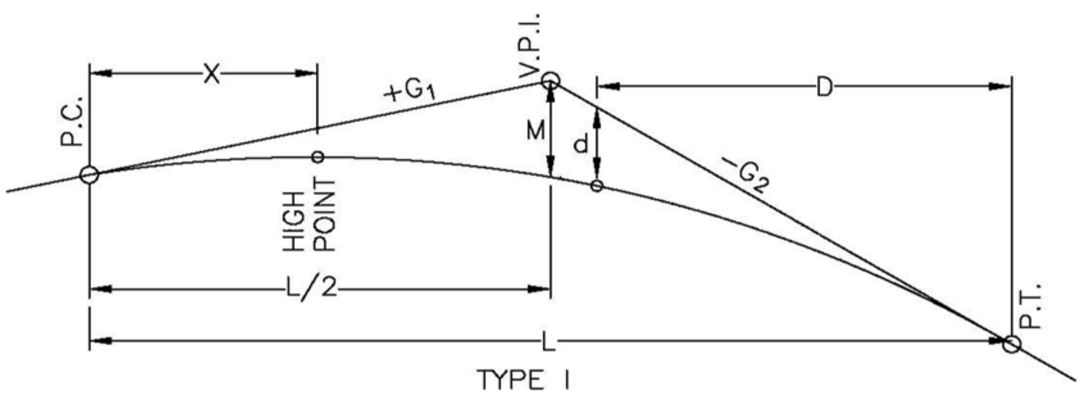
[d] Use only on urban streets.

Table 3-4 Relation of Maximum Grades to Design Speed

3.3.4 Vertical Curves

Properly designed vertical curves should provide adequate sight distance, safety, comfortable driving, good drainage, and pleasing appearance.

Vertical curves are parabolic. Figure 3-14 gives the necessary mathematical relations for computing a vertical curve, at either crests or sags.



TYPE I
CREST VERTICAL CURVE

L = LENGTH OF CURVE IN STATIONS

G_1 AND G_2 = GRADES IN FEET PER 100 FEET(%)

X = DISTANCE FROM THE P.C. OF THE CURVE IN STATIONS

d = DISTANCE IN FEET

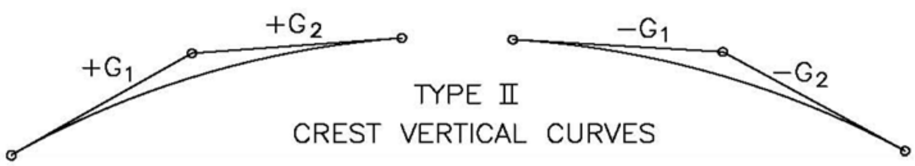
D = DISTANCE IN STATIONS

$$M = \frac{L\,(\text{ALGEBRAIC DIFFERENCE IN GRADES})}{8}$$

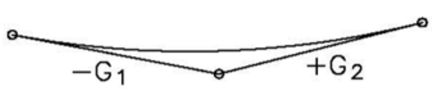
$$d = \frac{(\text{ALGEBRAIC DIFFERENCE IN GRADES})}{2L} \times D^2 \quad \text{OR} \quad d = 4M\left[\frac{D}{L}\right]^2$$

TO DETERMINE HIGH POINT
(OR LOW POINT ON SAG VERTICAL CURVES) USE THE FOLLOWING:

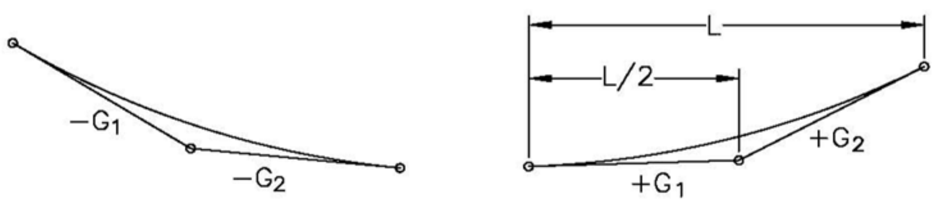
$$X = \frac{G_1 \times L}{(\text{ALGEBRAIC DIFFERENCE IN GRADES})}$$



TYPE II
CREST VERTICAL CURVES



TYPE III
SAG VERTICAL CURVE



TYPE IV
SAG VERTICAL CURVES

Figure 3-14 Vertical Curves

Minimum lengths of crest vertical curves are controlled by stopping sight distance requirements (see Table 3-1 and section 3.1.5.1).

Minimum lengths of sag vertical curves are typically controlled by headlight sight distance that should be approximately the same as stopping sight distance. In areas sufficiently lit where headlight sight distance is not a limitation, the passenger comfort factor can be used to determine the minimum length of sag vertical curve. Sag vertical curve lengths satisfying the comfort factor are approximately 50 percent of that needed to satisfy headlight sight distance criterion. Equation 3-51 (page 3-160) of the *PGDHS* **(1)** is the expression used to determine the minimum length of sag vertical curve needed to satisfy the comfort factor.

Vertical curves are not required where algebraic grade difference is less than 0.20 percent. In rural applications, the minimum length of vertical curves on main roadways, both crest and sag, should be 300 feet. For other applications, the minimum length should be about three times the design speed.

Vertical curves that have a level point and flat sections near their crest or sag should be evaluated for drainage where curbed pavements are used. Values of K = 167 (K=L/A where L is the length of curve in feet, and A is the algebraic difference in grade) or greater should be checked for drainage. K value is a coefficient by which the algebraic difference in grades (A) may be multiplied to determine the length in feet (L) of the vertical curve that will provide minimum sight distance.

Also, vertical curves that are long and flat may develop poor drainage at the level section. This difficulty may be overcome by adjusting the flow line of the ditch section.

3.3.5 Climbing Lanes

On long, steep grades, a climbing lane for the slow-moving vehicles may be required. Criteria for establishing the need for such lanes are usually based on traffic volume, capacities, percent of trucks, grades, speeds, and level of service. Because of many variables, no set of conditions can be properly described as typical. A detailed analysis should be made wherever climbing lanes are being considered. A discussion of the analytical approach to be followed is presented in the *PGDHS* **(1)**.

The following three criteria, reflecting economic considerations, should be satisfied to justify a climbing lane:

1. Upgrade traffic flow in excess of 200 vehicles per hour.

2. Upgrade truck flow rate in excess of 20 vehicles per hour.

3. One of the following conditions exist:

    - A 10 mph or greater speed reduction is expected for a typical heavy truck.
    - Level-of-service E or F exists on the grade.
    - A reduction of two or more levels of service is experienced when moving from the approach segment to the grade.

The width of the climbing lane should be the same width as the regular traffic lanes. The beginning of the climbing lane should be preceded by a tapered section with a taper ratio of 25:1, and at least 150 feet long.

Desirably, the shoulder on the outer edge of a climbing lane should be as wide as the shoulder on the normal two-lane section. Conditions, however, may dictate otherwise, particularly when the climbing lane is added to an existing highway. A usable shoulder of 4 feet wide or greater is acceptable.

The ideal design would be to extend the climbing lane to a point beyond the crest, where a typical truck could attain a speed that is within 10 mph of the speed of the other vehicles with the desirable speed being at least 40 mph, approximately at level of service D. Even this may not be practical in many instances because of the unduly long distance required for trucks to accelerate to the desired speed. For such a condition, a practical point to end the added lane is where the truck can return to the normal lane without undue hazard. In particular, this would be feasible where the sight distance becomes sufficient to permit passing with safety when there is no oncoming traffic or, preferably at least 200 feet beyond this point. In addition, a corresponding length of taper should be provided to permit the truck to return to the normal lane.

For example, on a highway where the safe passing sight distance becomes available 100 feet beyond this point, the truck lane should extend 100 feet:

- Plus 200 feet or 300 feet beyond the crest,
- Plus an additional length for taper, preferably at a ratio of 50:1 but with a taper length of at least 200 feet.

Figures 3-24 and 3-25 of the *PGDHS* **(1)** show the relationship between rate and length of grade for several reductions in speed. The 10 mph speed reduction curve is used as the design guide.

The method for determining passing lane location is described in section 3.4.3 (page 3-125) of the *PGDHS* **(1)**.

On steep two-lane downgrades, where trucks must reduce their speeds substantially below those of passenger cars, it may be desirable to provide an additional lane.

3.3.6 Passing Lanes

Passing lanes can be added on two-lane highways to improve traffic operation on sections of lower capacity and on lengthy sections (6 to 60 miles) where there are inadequate passing opportunities.

The logical location for a passing lane is where passing sight distance is restricted, but adequate sight distance should be provided at both the add and drop lane tapers. A minimum sight distance of 1000 feet on the approach to each taper is recommended. The selection of the location should consider the location of intersections and high-volume driveways as well as physical constraints such as bridges and culverts that could restrict provision of a continuous shoulder.

Use the following design procedure to identify the need for passing sections on two-lane highways:

1. Design horizontal and vertical alignment to provide as much of the highway as practical with passing sight distance. See Passing Sight Distance column in Table 3-1.

2. Where the design volume approaches capacity, recognize the effect of lack of passing opportunities in reducing the level of service.

3. Determine the need for climbing lanes.

4. Where the extent and frequency of passing opportunities made available by application of Criteria 1 and 3 are still too few, consider the construction of passing lane sections.

Passing lane sections should be sufficiently long to permit several vehicles in a line behind a slow-moving vehicle to pass before returning to the normal cross-section of two-lane highway. The minimum length, excluding tapers, should be 1000 feet. A lane added to improve overall traffic operations should be long enough, over 0.3 mile, to provide a substantial reduction in traffic platooning, see Table 3-5, below.

One-Way Flow Rate (veh/h)	Passing Lane Length (mi)
100-200	0.50
201-400	0.50-0.75
401-700	0.75-1.00
701-1200	1.00-2.00

Table 3-5 [Table 3-31 of the *PGDHS* (1)]Optimal Passing Lane Lengths for Traffic Operations Efficiency

The transition tapers at each end of the added lane section should be designed to encourage safe and efficient operation. The lane drop taper should be computed from the *MUTCD* **(2)** formulas below.

For S ≥ 45 mph, $$L = WS$$ [3-24]

For S < 45 mph, $$L = \frac{WS^2}{60}$$ [3-25]

Where:

L = Length of taper, ft
W = Width, ft
S = Speed, mph

The recommended length for the lane addition taper is half to two-thirds of the lane drop length. The transitions should be located where the change in width is in full view of the driver.

3.3.7 2+1 Roadways

The 2+1 roadway concept has been found to improve operational efficiency and reduce crashes for select two-lane highways. The 2+1 concept provides a continuous three lane cross section with alternating passing lanes, see Figure 15. This configuration may be suitable for corridors with traffic volumes higher than can be served with isolated passing lanes, yet not high enough to require a consistent four lane cross section.

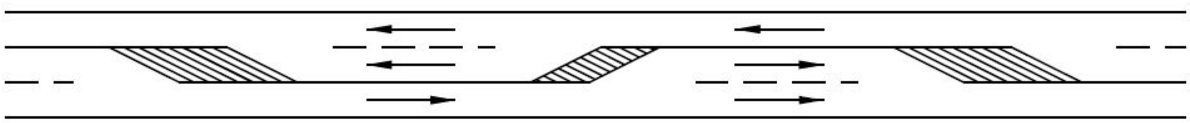


Figure 15 [Figure 3-33 of the *PGDHS* (1)] Schematic for 2+1 roadway

A 2+1 roadway generally operates two levels of service higher than a conventional two-lane road serving the same volume. 2+1 roadways should not generally be considered where the volume exceeds 1,200 vehicles per hour in one direction. 2+1 roadways should be used on level or rolling terrain; mountainous terrain or steep grades should consider climbing lanes as an alternative. Intersection locations should be considered when determining passing locations to minimize turning movements within passing lanes or to provide dedicated left turn lanes at intersections.

3.3.8 Turnouts

It will not always be economically feasible to provide passing lanes or desirably wide shoulders continuously along the highway through deep rock cuts or where other conditions limit the cross section width. In such cases, consideration should be given to use of intermittent sections of shoulder or turnouts along the highway. Such turnouts provide an area for emergency stops and also allow slower moving vehicles to pull out of the through lane to permit following vehicles to pass.

Turnouts should be located so that approaching drivers will have a clear view of the entire turnout in order to determine whether the turnout is available for use. Consider sight distance for vehicles re-entering the road. Refer to Table 3-32 of the *PGDHS* **(1)** for recommended lengths of turnouts including taper.

3.4 COMBINATION OF HORIZONTAL AND VERTICAL ALIGNMENT

To avoid the possibility of introducing serious hazards, coordination is required between horizontal and vertical alignment. Particular care must be exercised to maintain proper sight distance. Where grade line and horizontal alignment will permit, it is desirable to superimpose vertical curves on horizontal curves. This reduces the number of sight distance restrictions and makes changes in the profile less apparent, particularly in rolling country. Care should be taken, however, not to introduce a sharp horizontal curve near a pronounced summit or grade sag. This is particularly hazardous at night.

Horizontal curvature and profile grade should be made as flat as possible at highway intersections.

On divided highways, variation in the width of median and the use of separate profiles and horizontal alignment should be considered to derive the design and operational advantages of one-way roadways.

For additional information, see Chapter 2 of this Guide and Figure 3-46 of the *PGDHS* **(1)**.

3.5 GUIDELINES FOR DESIGNING DETOURS

For the purpose of applying these guidelines, a detour is any temporary routing of traffic off its usual course, including the use of existing alternate routes or use of modified lane widths on the main roadway.

The following criteria guidelines are recommended when designing a detour. Items are those which must be addressed when requesting detour approval. This is not a design policy and circumstances may often justify departure from these guidelines. For further reference, see the CDOT Form 518.

3.5.1 Detour Design Speed

The design speed of a detour should be as close to the mainline operating speed as possible. Every effort should be made keep the speed differential within 10 mph so as not to affect the capacity, although in some cases a maximum of 15 mph or more may be considered. As truck traffic increases so should the emphasis on providing the lowest possible speed differential. An exception may be posted city streets. See section 3.5.7.

3.5.2 Detour Clear Zone

Use the criteria corresponding to the speed, geometry, and traffic of the existing highway for designing the detour. Detour culverts should be included in the clear zone analysis.

Portable barrier may often be the most cost-effective method of resolving detour clear zone problems.

3.5.3 Detour Typical Section

- **Lane Width**

  It is desirable to maintain the width of the main roadway, but if this is not practical, the following guidelines apply:

  A minimum lane width of 10 feet may be used if all of the following conditions are satisfied:

  - The truck annual average daily traffic (AADT) is less than 50.
  - The design speed is ≤ 45 mph.
  - No curves are greater than 7 degrees.

  If one or more of the above conditions fails, 11-foot or wider lanes should be used.

  If any of the following conditions apply, 12-foot lanes should be used:

  - Design speed of 55 mph or more.
  - The truck AADT is greater than 300.
  - The road is an arterial or on an arterial truck network system.

  If main roadway lanes are 11 feet, the detour may retain 11-foot width.

- **Shoulder Width**

  Desirable shoulder width is 4 feet. Two feet minimum is required.

3.5.4 Detour Barrier

Barrier will be required when any hazards exist within the clear zone including drop-offs or steep slopes. It may also be required for the protection of workers. Shoulder Dropoffs 3 inches or greater shall be mitigated within 24 hours.

When barrier is used it shall be installed at least 2 feet offset from edge of pavement with an appropriate distance from back of barrier to obstruction; see *CDOT Standard Plans - M & S Standards* **(3)**. If shoulders are not provided, it shall be installed 2 feet from the edge of the traveled lane. Where the situation allows, an offset of 4 feet from the traveled lane should be provided.

3.5.5 Detour Surfacing

An asphalt surface is usually functionally superior to gravel, although gravel may have economic and other advantages. Asphalt should be used if detour speed is over 40 mph or the detour will be used for three weeks or more. Consult Region Materials Engineer for detour pavement design. Refer to the *CDOT Project Development Manual* **(9)** for additional information.

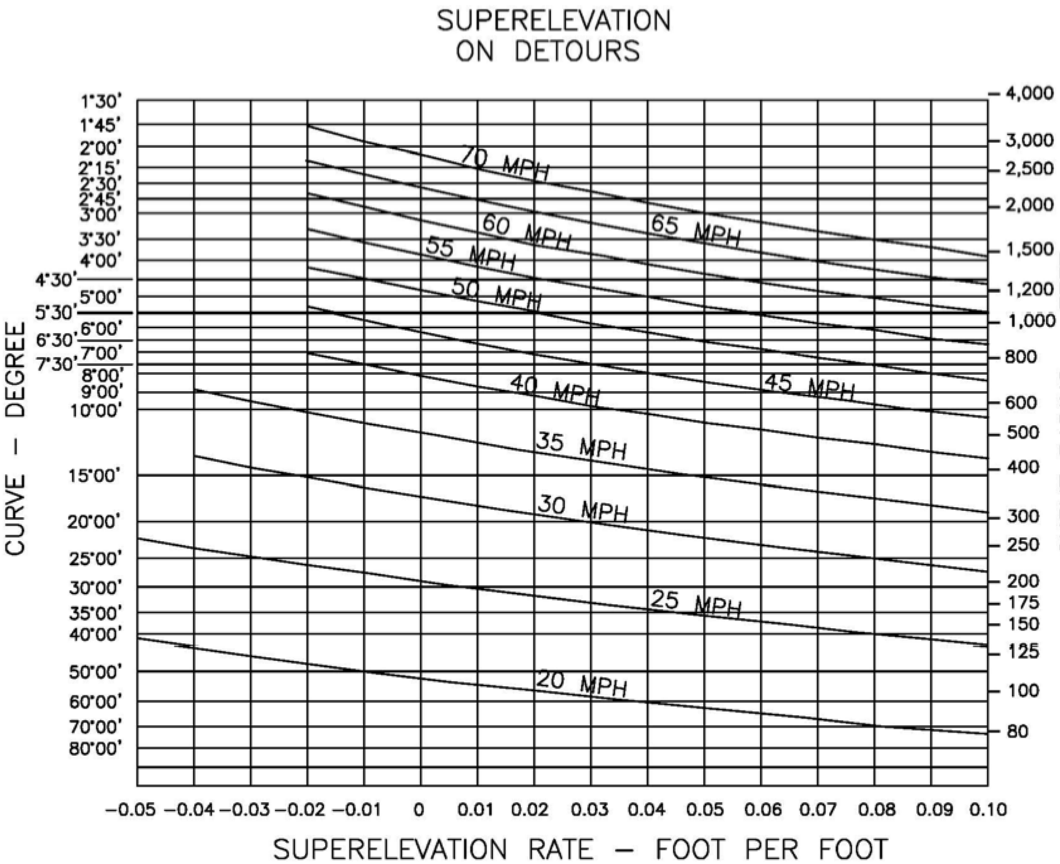
3.5.6 Detour Superelevation

Figure 3-16 gives the rate of superelevation to be used on detours during construction of culverts, bridge replacement or widening, or repairs when proper construction signing is in place.

The formula shown in Figure 3-16 is the same as used in *CDOT Standard Plans - M & S Standards* **(3)** Superelevation, which shows superelevation and curvature for various design speeds, except that the "e" value shown in *CDOT Standard Plans - M & S Standards* **(3)** is based on maximum driving comfort and safety, combined with a widely variable friction factor caused by adverse pavement surface and weather conditions.

By use of Figure 3-16 the designer can choose a combination of friction factor, superelevation, and curvature to meet required design speed without the necessity of building up an excessive amount of superelevation and runoff which must be removed after a short time.

Values on Figure 3-16 have been checked by the "Ball Bank Indicator" to determine the point of discomfort for safe speeds on curves.



SPEED	FRICTION FACTOR
20	0.240
25	0.208
30	0.180
35	0.162
40	0.150
45	0.145
50	0.140
55	0.135
60	0.130
65	0.127
70	0.120

e= SUPERELEVATION
f= FRICTION FACTOR
V= SPEED– MILES PER HOUR
R= RADIUS IN FEET

$$V = \sqrt{15(e+f)R}$$

Figure 3-16 Superelevation on Detours

3.5.7 Detour on Local Roads

When local roads are used in detour routing, the stabilization needs must be reviewed. If necessary, additional overlay should be placed to protect the structural integrity of the street.

All the above-listed design elements, including the information on "Detour Design Data," shall be specified in the plans.

The following should be considered:

- Inter-Governmental Agreement
- Weight Limits
- Noise
- Traffic
- Schools

3.5.8 Environmental Considerations for Detours

When designing detours, it is important to consider and mitigate any possible environmental impacts. These can include wetlands, archaeology or paleontology resources, hazardous waste, water quality or 4(f) involvement. These impacts may be avoided by the proper placement of the detour. For assistance in evaluating possible impacts, contact the Region Planning/ Environmental Section

3.5.9 Detour Transverse Underdrains

Transverse underdrains are those constructed perpendicular to roadway. See the *CDOT Drainage Design Manual* **(10)**.

3.6 OTHER ELEMENTS AFFECTING GEOMETRIC DESIGN

3.6.1 Drainage and Erosion Control

Consider the following:

- Collect water prior to transitioning superelevation to prevent sheet flow.
- Design and locate inlets to limit the spread of water on the traveled way to tolerable widths.
- Install extra inlets near low points of sag vertical curves to take any overflow from blocked inlets.
- Locate inlets just upgrade of pedestrian crossings.
- Address environmental issues such as erosion and sediments.
- Dikes in medians and on the edge of the road should comply with clear zone requirements.
- Coordinate NPDES issues with Region Planning and Environmental Manager [see the *CDOT Project Development Manual* **(9)**].

Also see the *CDOT Drainage Design Manual* **(10)**.

3.6.2 Rest Areas

Coordinate with the FHWA operations engineer. Refer to the AASHTO *Guide for Development of Rest Areas on Major Arterials and Freeways* **(11)** and Colorado “Rest Area Management & Maintenance Study."

3.7 LIGHTING

Consider the following:

- Coordinate with the Region Utility Engineer and the local utility company.
- Minimize light pollution in conformance with 24-82-902 Colorado Revised Statutes (CRS).
- Safety enhancement.

3.8 UTILITIES

Consider the following:

- Coordinate with the Region Utilities Engineer early and throughout the design process.
- Pothole to locate utilities as practical.
- Plot existing utilities in plan, profile, and cross sections to identify potential conflicts with design elements.
- Utility Notification Center of Colorado will not locate CDOT owned utilities; contact the Region Traffic Signal Supervisor.
- Utility relocation requirements should be compatible with construction phasing.
- An Inter-Governmental Agreement (IGA) may be necessary.

The clear zone dimensions to be maintained for a specific functional classification are discussed in the section 4.6.1 of the *PGDHS* **(1)**.

Utilities that are to cross or otherwise occupy the right of way of rural or urban freeways should conform to the AASHTO *A Policy on the Accommodation of Utilities Within Freeway Right-of-Way* **(12)**. Those on non-controlled access highways and streets should conform to the AASHTO *A Guide for Accommodating Utilities Within Highway Right-of-Way* **(13)**.

3.9 TRAFFIC CONTROL DEVICES

The development of traffic control plans is an essential part of the overall project design and may affect the design of the facility itself. See Chapter 20, “Traffic and Safety Engineering” and the *MUTCD* **(2)**.

3.10 NOISE BARRIERS

See Chapter 18.

REFERENCES

1. AASHTO. *A Policy on Geometric Design of Highways and Streets*, American Association of State Highway and Transportation Officials, Washington, D.C.: 2011.

2. U.S. Department of Transportation, Federal Highway Administration, *Manual on Uniform Traffic Control Devices for Streets and Highways (MUTCD),* Washington, D.C.: 2009.

3. CDOT. *CDOT Standard Plans - M & S Standards*, Colorado Department of Transportation, 2012.

4. Hickerson, Thomas F., *Route Location and Design*, McGraw-Hill Book Company, New York, N.Y., 1964.

5. AASHTO. *Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals*, American Association of State Highway and Transportation Officials, Washington, D.C.: 2015.

6. AASHTO. *Roadside Design Guide*, American Association of State Highway and Transportation Officials, Washington, D.C.: 2011.

7. AASHTO. *Standard Specifications for Highway Bridges*, American Association of State Highway and Transportation Officials, Washington, D.C.: 2002.

8. CDOT. *CDOT Bridge Design Manual*, Colorado Department of Transportation, 2018.

9. CDOT. *CDOT Project Development Manual,* Colorado Department of Transportation, 2013 (with revisions through 2016).

10. CDOT. *CDOT Drainage Design Manual*, Colorado Department of Transportation, 2017.

11. AASHTO. *Guide for Development of Rest Areas on Major Arterials and Freeways*, American Association of State Highway and Transportation Officials, Washington, D.C.: 2001.

12. AASHTO. *A Policy on the Accommodation of Utilities Within Freeway Right-of-Way*, American Association of State Highway and Transportation Officials, Washington, D.C.: 2005.

13. AASHTO. *A Guide for Accommodating Utilities Within Highway Right-of-Way,* American Association of State Highway and Transportation Officials, Washington, D.C.: 2005.

14. CDOT. *State Highway Utility Accommodation Code (2 CCR 601-18), Effective October 20, 2009.* Colorado Department of Transportation, 2009. [online http://www.sos.state.co.us/CCR/GenerateRulePdf.do?ruleVersionId=3222&fileName=2%20CCR%20601-18]

CHAPTER 4
CROSS SECTION ELEMENTS

List of Tables

List of Figures

CHAPTER 4
CROSS SECTION ELEMENTS

4.1 PAVEMENT

4.1.1 Surface Type

The selection of pavement type is determined by the volume and composition of traffic, soil characteristics, weather, performance of pavements in the area, availability of materials, energy conservation, the initial cost and the overall annual maintenance and service life cost. The structural design of pavements is not included in this chapter, but may be found in the CDOT *Pavement Design Manual* **(1)**.

4.1.2 Cross Slope

Design of the pavement cross slope is often a compromise between the need for reasonably steep cross slopes for drainage and relatively flat cross slopes for driver comfort. Pavement superelevation on curves is determined by the speed-curvature relationships given in Chapter 3 and *CDOT Standard Plans - M & S Standards* **(2)**.

Two-lane and wider undivided highways on tangents have a crown or high point at the centerline and slope downward at an even rate outward. On high and intermediate type undivided pavements, crowned at the center, the designated rate of cross slope is 2 percent.

An overlay may be matched to the existing cross-slope unless the safety assessment report indicates otherwise. The profile grade and pivot point are generally located at the pavement crown.

On divided highways, each one-way pavement has a unidirectional slope across the entire width of pavement. Except as allowed otherwise in 4.3.3. The tangent cross slope is 2 percent, with the profile grade and pivot point located at the inside edge of the inside travel lane. At intersections, interchange ramps or in unusual situations, the crown position may vary depending upon drainage or other controls.

To avoid excessively close inlet spacing, hydroplaning, and other drainage problems associated with flat pavement slopes and wide typical cross sections, Superelevation transitions and vertical curve lengths should be minimized in wide typical cross sections to reduce flat areas that may accumulate water.

4.1.3 Skid Resistance

CDOT's practice for concrete pavements is to use longitudinal tining. Tining is required for design speeds of 40 mph and greater.

4.1.4 Traveled Lane Texturing

Traveled Lane rumble strips should be considered for locations with unanticipated changes to the roadway:

- In advance of a stop sign or lane reduction on a rural multilane or a divided controlled-access arterial and
- Other locations as determined by the Region Traffic Engineer.

See *CDOT Standard Plans – M & S Standards M-614-1* **(2)** for details.

4.2 LANE WIDTHS

The basic traveled lane is 12 feet wide for both two-lane and multilane roads. In specific cases, traveled lanes 11 or 10 feet wide may be used (see Table 4-1).

4.3 SHOULDERS

4.3.1 General Characteristics

A shoulder is the portion of the roadway contiguous with the traveled way for accommodation of stopped vehicles, for emergency use, for bicycle use, and for lateral support of subbase, base, and surface courses.

4.3.2 Width of Shoulders

For purposes of this Guide, the shoulder shall be the minimum continuous usable width of shoulder which provides an all-weather surface. The width will vary from 4 to 12 feet, depending on the roadway design type. Shoulders should be paved full-width in accordance with the functional or physical characteristic of the roadway. The shoulder width may be changed where speed-change lanes, climbing lanes, or curb and gutter are used. See Chapter 14 for further information when designing bicycle paths.

4.3.3 Shoulder Cross Sections

In most cases, the shoulder slope should be the same as the crown slope of the traveled way. However, in sag vertical curves in tangent horizontal alignments with narrow shoulder widths or where inlet spacing becomes excessive, the shoulder cross slope should be steepened 1 percent more than the adjacent lane cross slope to increase the hydraulic carrying capacity of the shoulders. The designer also should consider breaking the shoulder in the opposite direction to reduce water draining across the road and causing icy conditions. Shoulder cross slope may have an algebraic difference of 5 percent from the traveled way cross slope. Greater algebraic differences require rounding outside the travel lane.

4.3.4 Shoulder Stability

Shoulders should be paved full-depth in accordance with the functional or physical characteristic of the roadway. At certain locations, erosion of topsoil or shouldering materials adjacent to the pavement edge results in hazardous conditions for the motorist and a reduction in lateral support for the pavement. The use of a safety edge is required in all projects having roadway pavement. Vertical pavement edge drop-off on highways has been linked to many serious crashes when errant vehicles attempt to steer back onto the roadway. Instead of a vertical drop-off, the safety edge shapes the edge of the pavement to 30 to 35 degrees. Research has shown this is the optimal angle to allow drivers to re-enter the roadway safely. See Project Special Detail D-614-1 of the *CDOT Standard Plans - M & S Standards* **(2)** for more information**.**

4.3.5 Shoulder Contrast

Distinguishing paved shoulders from the mainline pavement is recommended to discourage the use of the shoulder as a travel lane and provide guidance and warning to drivers. This can be accomplished by pavement markings and differences in shoulder surface texture.

Shoulder texture treatments that provide an audible or vibrational warning to errant drivers have proven effective in keeping traffic off the shoulder and reducing accidents on long tangent or monotonous highway sections with a history of run-off -the-road accidents. Rumble strips may also be utilized to minimize run-off-the-road accidents and shall comply with the current *CDOT Standard Plans - M & S Standards* **(2)**.

Rumble strips should always be considered for use in rural areas where flat or rolling terrain with long tangents and relatively flat curvature is predominant, encouraging driver inattentiveness or drowsiness. Under these circumstances, shoulder grooving is highly recommended.

If there is any doubt on the use of rumble strips, then further analysis should be done, taking into consideration run-off -the-road single vehicle accidents, bicycle usage, and other pertinent factors supporting use or non-use. Suggested designs to accommodate rumble strips and a bicycle facility are found in Chapter 14.

Other textures or methods, such as coarse chip seals, may be used in lieu of shoulder grooving on an experimental basis. Chip seals are not recommended in areas with regular bicycle traffic.

4.3.6 Turnouts

See section 3.3.8.

4.4 TYPICAL SECTIONS

The dimensions of a typical cross section depend upon a number of features that vary with the type of roadway. Geometric design standards as used by CDOT are shown in Table 4-1. The design may be adjusted for specific conditions as indicated in the appropriate chapter of this guide or in the chapter of *PGDHS* **(3)** for the specific classification of roadway.

CDOT includes Z slopes in typical sections. The Z slopes, which slope gently away from the edge of the pavement, provide for safety, drainage, snow storage, sign placement, and rockfall containment and shall be included in the design. The typical section also indicates the locations of the following points:

- Hinge Point: The point on the subgrade directly below the edge of the pavement from which the subgrade slopes downward to the point of slope selection.
- Point of Slope Selection: The point at the toe of the Z slope that intersects with the subgrade. The point of slope selection is the point at which the embankment or excavation begins to slope away from the roadway prism.

- Profile Grade: The point on the pavement surface, defined by its location on the vertical alignment of the roadway, from which all other points and slopes on the cross section are determined.
- Pivot Point: The point on the pavement surface about which the cross slope of the roadway is rotated to effect superelevation. The pivot point may be at the same location as the profile grade.

Geometric Design Type[1]	No. of Lanes	Lane Width (ft.)	Shoulder Width (min)[2] (ft.)		ROW Width (ft.)				Bridges and Grade Separations	
						Suggested Minimum				
			Outside	Inside	Desir.	With Frontage Road	Without Frontage Road	Desir. Access Control	Design Load	Clear Roadway Width
Type AA	6[3]	12	Freeways 10[4] Arterials 10	Freeways 10[4] Arterials 8[4]	300	275	175	Full	HS 20-44[5]	See Note 6
Type A	4[3]	12	10	4	300	250	150	Full[7]	HS 20-44[5]	See Note 6
Type B	2[3,8]	12	8 10[9]		250	250	150	See Note 7	HS 20-44[5]	See Note 6
Type C	2	11 12	6[10] 6[10]		120		60	See Note 7	HS 20-44[5]	See Note 6
Type D	2	10 11	4 4		100		60	See Note 7	HS 20-44[5]	See Note 6

1. "Types" refers to details shown on Figures 4-1 through 4-5.
2. Shoulder widths may not apply when roadway has curb and gutter, speed-change lanes, etc.
3. See *Highway Capacity Manual* **(4)**.
4. Where the DDHV for truck traffic exceeds 250 veh/h, a paved shoulder width of 12 feet should be considered.
5. Alternate loadings for two 24,000-pound axles shall be used where applicable on the Interstate.
6. Bridge widths will be determined in accordance with requirements set forth in the latest revision of the *PGDHS* **(3)**, *AASHTO Standard Specifications for Highway Bridges* **(5)** and *CDOT Standard Plans - M & S Standards* **(2)**. Special cases will be subject to consideration by the CDOT Staff Bridge Engineer.
7. To be decided on an individual project basis. Interstate requires full access control.
8. Climbing lanes should be provided in accordance with 3.4.5 of this Guide.
9. Minimum 10' shoulder should be used when DHV exceeds 400, except in mountainous terrain where the 8' minimum shoulder will remain standard for DHV over 400.
10. Minimum 3' paved shoulder with 3' gravel shoulder.

For median widths, see chapter for the specific classification of roadway
For maximum grades, see chapter for the specific classification of roadway.
For minimum radius of curve, refer to the *CDOT Standard Plans - M & S Standards* **(2)** and 3.2.3.2 of this Guide.

Table 4-1 Geometric Design Standards

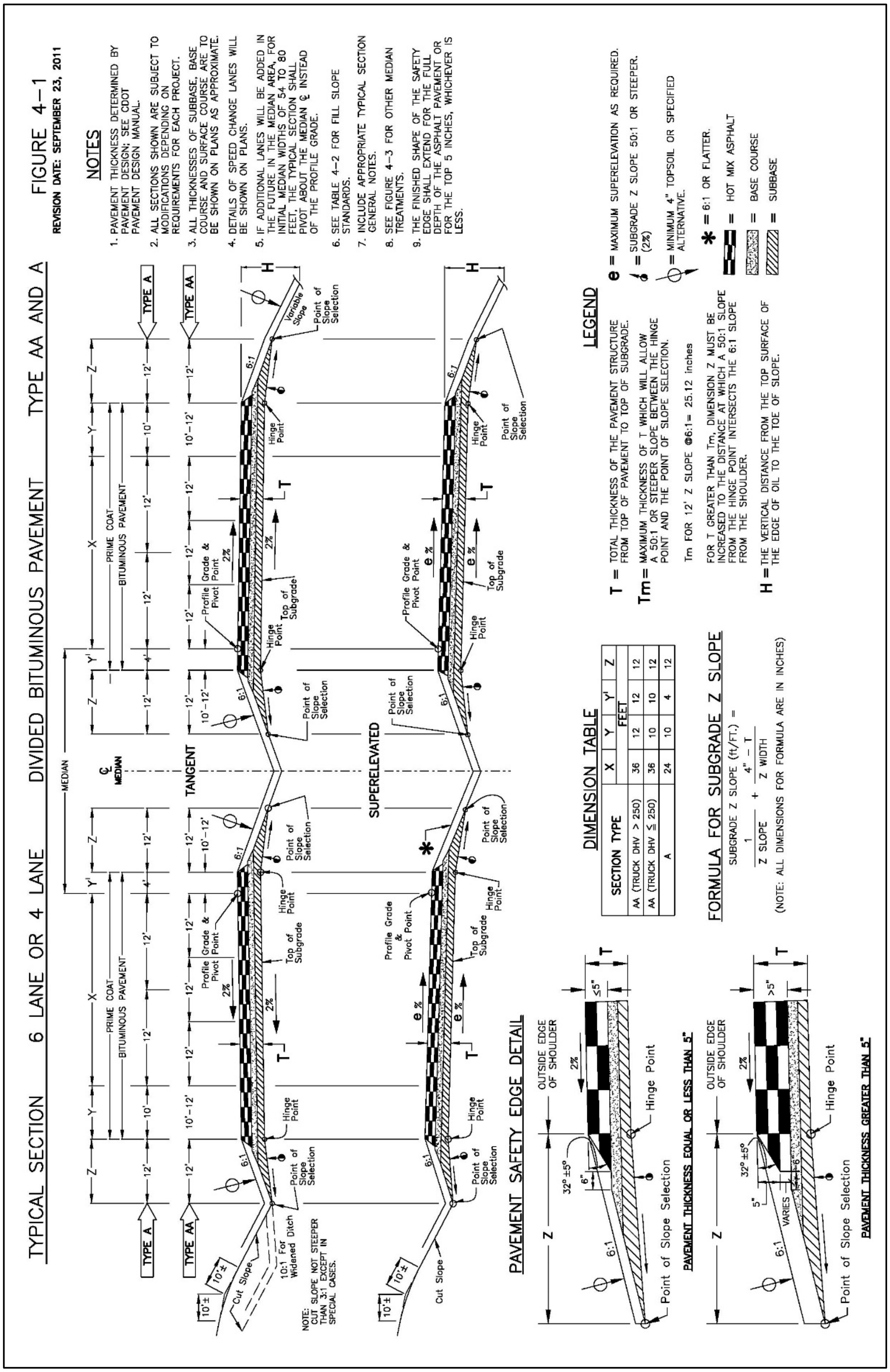


Figure 4-1 Typical Sections for Divided Bituminous Pavement

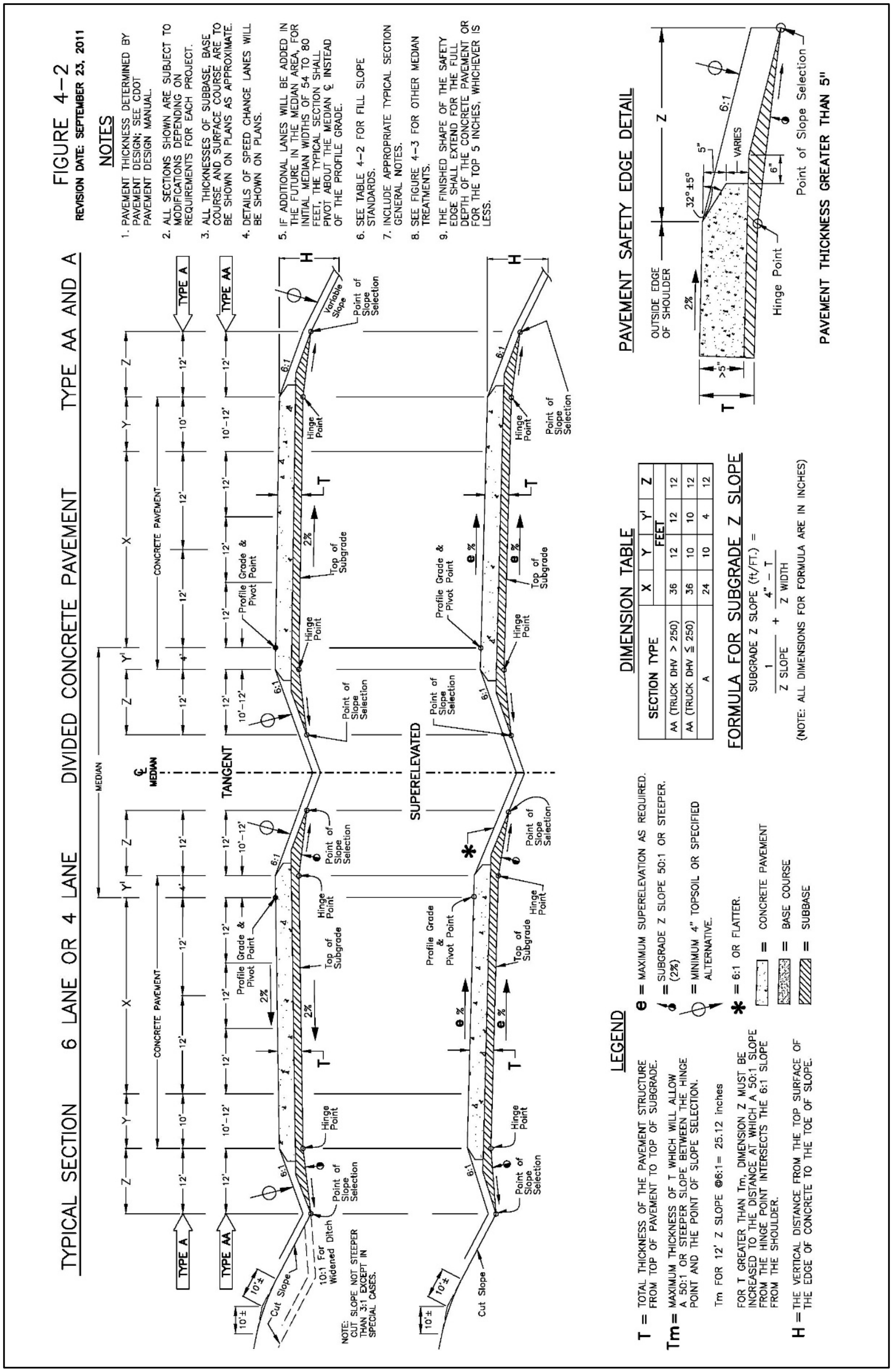


Figure 4-2 Typical Sections for Divided Concrete Pavement

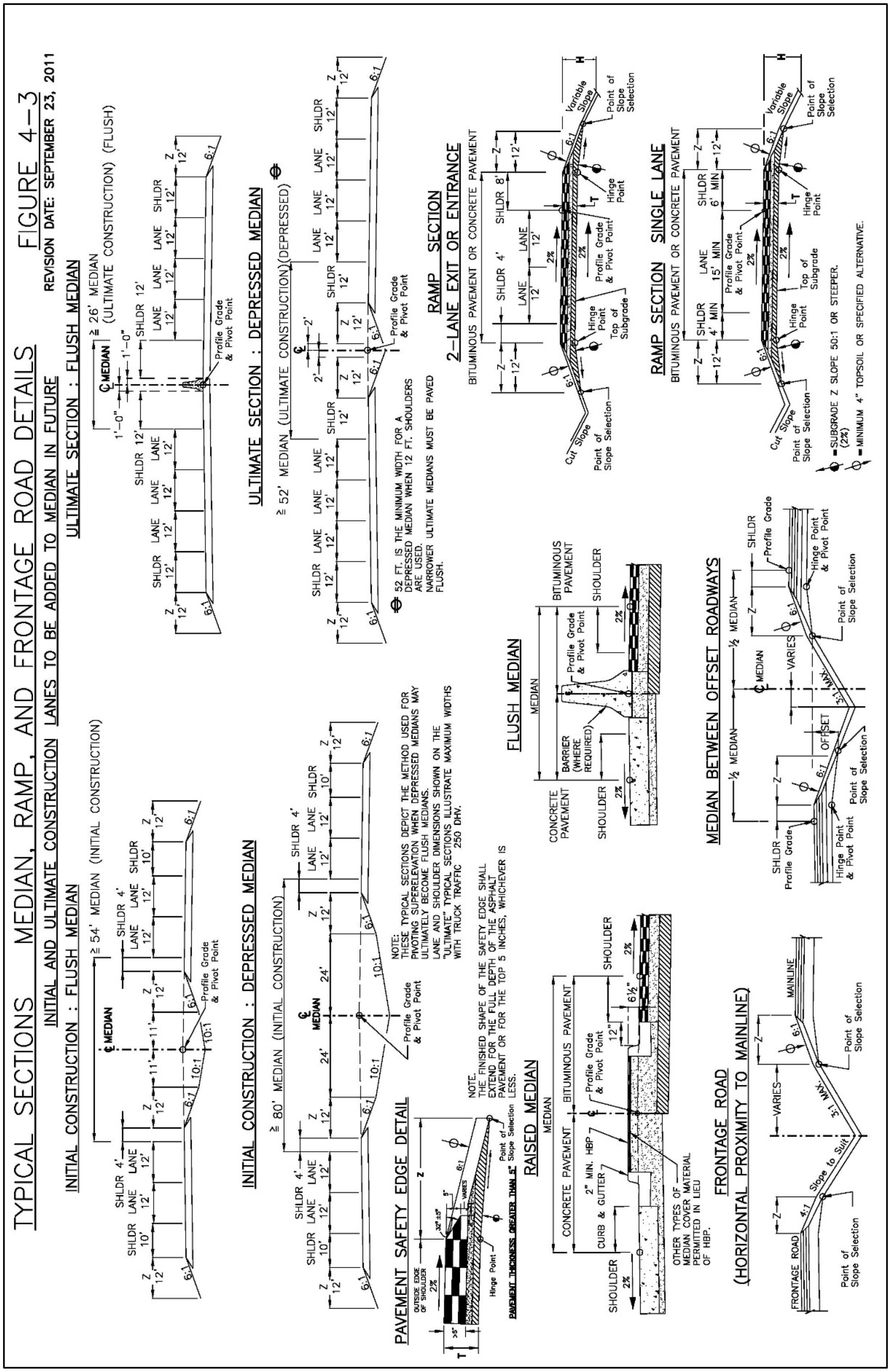


Figure 4-3 Typical Sections for Medians, Ramps, and Frontage Roads

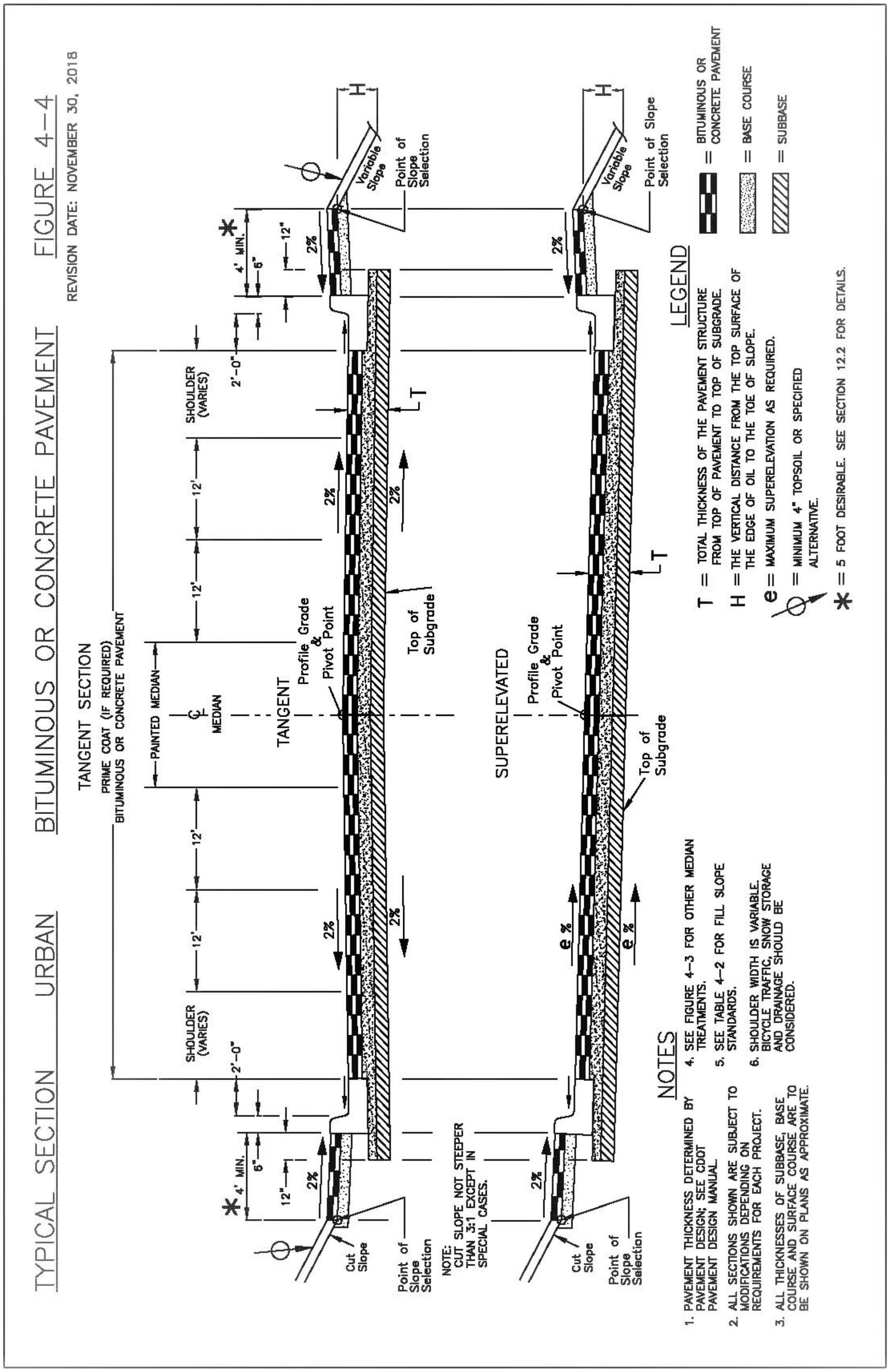


Figure 4-4 Typical Sections for Urban Bituminous or Concrete Pavement

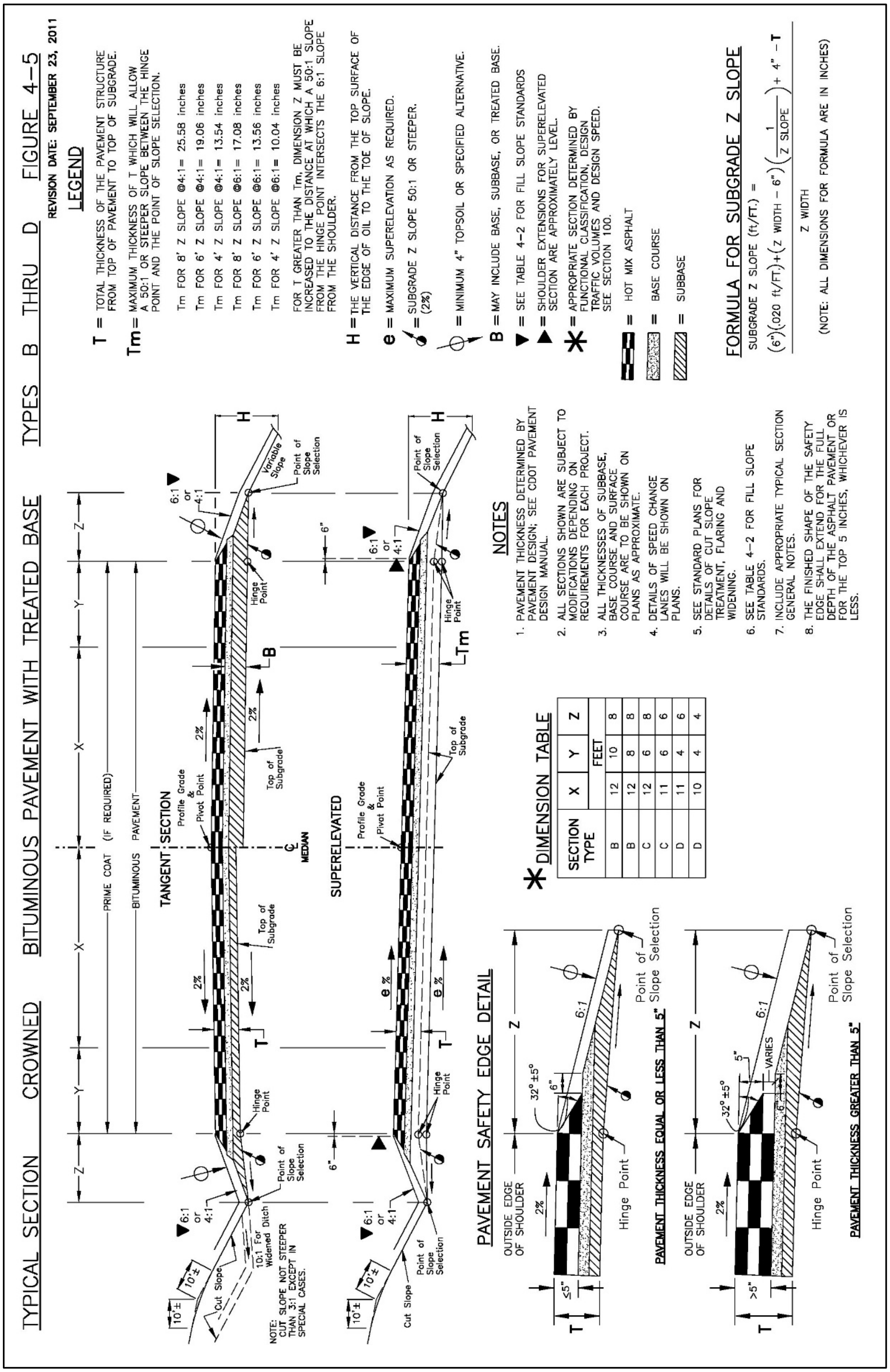


Figure 4-5 Typical Section for Crowned Bituminous Pavement

4.5 HORIZONTAL CLEARANCE TO OBSTRUCTIONS

The term "clear zone" is used to designate the unobstructed, relatively flat area provided beyond the traveled way for the recovery of errant vehicles. The traveled way does not include shoulders.

The width of the clear zone is influenced by the traffic volume, speed, and embankment slopes as discussed in the *AASHTO Roadside Design Guide* **(6)**. The Guide should be used as a reference for determination of clear zone for freeways, rural arterials, and high-speed rural collectors. For low-speed, low volume rural collectors and local roads, a minimum of 10 feet should be provided.

For urban arterials, collectors and local streets where curbs are utilized, space for clear zones is generally restricted. A minimum offset distance of 18 inches should be provided beyond the face of the curb to the face of the obstruction, except where *CDOT Standard Plans - M & S Standards* **(2)** specify 24 inches, with wider clear zones provided where practical or where existing hazards require additional clear zone. The designer should consider pole foundation location to avoid interference with back of curb. Where shoulders are provided rather than curbs, a clear zone commensurate with the *AASHTO Roadside Design Guide* **(6)** should be provided. Because of large-vehicle overhang, 3 feet from face of curb is considered minimum clearance in intersections.

4.6 CURBS

Curbs serve the following purposes:

- Drainage control
- Pavement edge delineation
- Right of way reduction
- Aesthetics
- Delineation of pedestrian walkways
- Reduction of maintenance operations
- Assistance in orderly roadside development.

Use of curbs on high speed highways is discouraged in the interest of safety. In urban situations, curb is common; in intermediate speed situations, judgment must be exercised.

Curb should be used where required for proper drainage and where needed for channelization or access control.

Curbs may be sloping or vertical types. Vertical curbs shall be used only on roadways where speed is 45 mph or less, with sloping curb being used where speeds are greater than 45 mph. Curbs should not be used in front of the face of traffic barriers as the curb vaulting action interferes with the proper action of the barrier. See *CDOT Standard Plans - M & S Standards* **(2)**. Vertical curbs should not be used along freeways or other high-speed arterials, but if a curb is needed, it should be of the sloping type and should not be located closer to the traveled way than the outer edge of the shoulder. Refer to the *CDOT Drainage Design Manual* **(7)** for minimal longitudinal slope requirements.

One-foot gutter is used with the curb where drainage control is not required; e.g., medians and islands. Sloping curb may be used on islands and medians where crossing is not a problem. Vertical curb should be used wherever random crossing or U-turn movements create a hazard.

Areas which require drainage control should use curb with 2-foot gutter. Cross slope of curb varies depending on whether the water is to flow toward or away from the curb. See *CDOT Standard Plans - M & S Standards* **(2)**. Inlets and storm drains are favored over using valley gutter for carrying drainage across intersections. Radii of curbs in intersections are generally dictated by the type of highway being served.

Normally, the curb and gutter are not considered to be part of the travel lane width; however, there may be exceptions in urban areas. A 2-foot offset from the edge of travel way to the face of outside curb is preferred because placement of the curb may affect driver perception of the roadway, causing shying away from the curb.

Certain special circumstances occur with curb:

- Special curb types such as curb without gutter or bituminous curbing may be required.
- 10-foot transitions between curb cross sections may be necessary.

Further specific information on curbs may be found in the *CDOT Standard Plans - M & S Standards* **(2).**

4.7 DRAINAGE CHANNELS AND SIDESLOPES

4.7.1 General Considerations

Modern highway drainage design should incorporate safety, good appearance, control of pollutants, and economy in maintenance and construction. The above may be direct benefits of using flat sideslopes, broad drainage channels, and liberal warping and rounding. These features avoid obsolescence, improve appearance, and invite favorable public reaction.

The interrelationship between the drainage channel and the sideslopes is important for safety because of their great influence on the sequence of events that can occur when a vehicle leaves the traveled way.

4.7.2 Drainage Channels

Drainage channels perform the vital function of collecting and conveying surface water from the highway right of way. Drainage channels, therefore, should have adequate capacity for the design runoff, should provide for unusual storm water with minimum damage to the highway, and should be located and shaped to avoid creating a hazard to traffic.

Design flows and channel capacities can be determined based upon the *CDOT Drainage Design Manual* **(7)**. Side ditches should be used in all cut sections. Steep-sided channels are more desirable from a hydraulic point of view, but hydraulic performance must be evaluated in light of potential hazards steep slopes pose to errant vehicles. Channels shall be designed in accordance with the *AASHTO Roadside Design Guide* **(6)**.

The depth of channel should be sufficient to remove the water without saturation of the pavement subgrade. The depth of water that can be tolerated, particularly on flat channel slopes, depends upon the soil characteristics.

4.7.3 Sideslopes

Sideslopes should be designed to ensure the stability of the roadway and to provide a reasonable opportunity for recovery for an out-of-control vehicle. Three regions of the roadside are important when evaluating the safety aspects: the top of the slope (hinge point), the foreslope, and the toe of the slope (intersection of the foreslope with level ground or with a backslope, forming a ditch). In many situations, the toe of the slope is within the clear zone and the probability of reaching the ditch is high, in which case safe transition between fore and backslopes should be provided. Figure 4-6, General Cross Sectional Information, illustrates these three regions.

Rounded slopes reduce the chances of an errant vehicle becoming airborne, thereby reducing the hazard of encroachment and affording the driver more control over the vehicle. Foreslopes steeper than 4:1 are not desirable because their use severely limits the choice of backslopes. Slopes 3:l or steeper are recommended only where site conditions do not permit use of flatter slopes. Clear runout space at the base of non-recoverable slopes is desirable. When foreslopes steeper than 3:1 must be used, consideration should be given to the use of a roadside barrier.

Normally, backslopes should be 3:l or flatter to make it easier for motorized equipment to be used in maintenance. In developed areas, sufficient space may not be available to permit the use of desirable slopes. Backslopes steeper than 3:1 should be evaluated with regard to soil stability and traffic safety.

Design of a safe roadside depends upon design speed, traffic volumes, horizontal and vertical alignment of the roadway, type of highway, and other factors. For a thorough discussion of safety in design of side slopes, highway clear zones, and drainage channels the designer should refer to the *AASHTO Roadside Design Guide* **(6)**.

4.7.4 Cut Slope Standards

Cut slopes should not be steeper than 3:1 unless material is encountered which will stand on steeper slopes. Flatter slopes should be used in shallow cuts or in soils highly susceptible to erosion.

4.7.5 Fill Slope Standards

Fill slopes are determined by a combination of terrain, height, and template type. The other chapters have specific information on desired fill slopes and CDOT practices for different highway types. Where 3:1 slopes or steeper are used, a comparison of costs of these slopes with any guardrail required vs. flatter slopes should be made. See the *AASHTO Roadside Design Guide* **(6)** for guardrail guidelines.

Table 4-2 gives the standards for fill slopes to be used for different types of highways. Flatter slopes should be used in soils highly susceptible to erosion.

Highway Type	* H	Terrain	
		Plains	Rolling and Mountainous
		Slope Ratio**	
4 or more lanes (Z=12' @ 6:1)	≤ 4' > 4' to 10' > 10' to 15' > 15'	Z, then 6:1 Z, then 4:1 Z, then 4:1 Z, then 3:1	Z, then 4:1 Z, then 4:1 Z, then 3:1 Z, then 3:1
‡2 lane (Z=8' @ 6:1 or 6' @6:1 or 4' @ 6:1)	≤ 4' > 4' to 10' > 10' to 15' > 15'	Z, then 6:1 Z, then 4:1 Z, then 4:1 Z, then 2:1	Z, then 4:1 Z, then 4:1 Z, then 3:1 Z, then 2:1

* H is the vertical distance between outside edge of top layer of pavement and catch point where fill meets natural ground.
Slopes 3:1 or steeper should be reviewed for safety and guardrail warrants See Figures 4-1 to 4-5 for determination of Z width.
**May be steeper in special cases.
‡ In constrained situations on a 2 lane roadway, the Z slope may be constructed as steep as 4:1.

Table 4-2 Fill Slopes

4.7.6 Clearance from Slope to Right of Way Line

The minimum clearance from the right of way line to the catch point of a cut or fill slope should be 10 feet for all types of cross sections, but the desirable clearance is 20 feet. Access for maintenance activities should be considered.

4.7.7 Slope Benches

The necessity for benches, their width, and vertical spacing should be determined only after an adequate geotechnical materials investigation has been completed. Contact the Materials and Geotechnical Branch.

For ease of maintenance, a 20-foot width of bench is satisfactory. Benches slope approximately 20:1 towards the roadway to prevent ponding of moisture behind the bench, thus creating additional slip plane problems. Benches should be constructed to blend with geologic strata rather than conforming to any set grade.

4.7.8 Cut Slope Treatment

The tops of all cut slopes should be rounded where the material is other than solid rock. A layer of earth overlying a rock cut also should be rounded. See the Slope Rounding details on the typical sections, Figures 4-1, 4-2, and 4-5 and subsection 203.05 of *CDOT Standard Specifications for Road and Bridge Construction,* **(8)**.

4.8 ILLUSTRATIVE GENERAL CROSS SECTIONS

Figure 4-6 illustrates typical combinations of the highway elements: lanes, shoulders, side-drainage channels, sidewalks, curbs, and sideslopes.

4.8.1 Normal Crown Sections

Figure 4-6 shows cross sections commonly used in CDOT highway practice, undivided and divided highways. Shoulder widths and Z slopes are included on both the fill and cut sections. The embankment or fill sections on the right side of the sections are composed of the Z slope and fill slopes.

The drainage channels shown on the left side of the sections are formed by the Z slope on the roadway side and the cut slope or backslope on the outer side. The Z slope and backslope combination should be selected to produce a cross section that can be safely traversed by an errant vehicle. Any hazardous object should be located outside of the clear zone as discussed previously in this section.

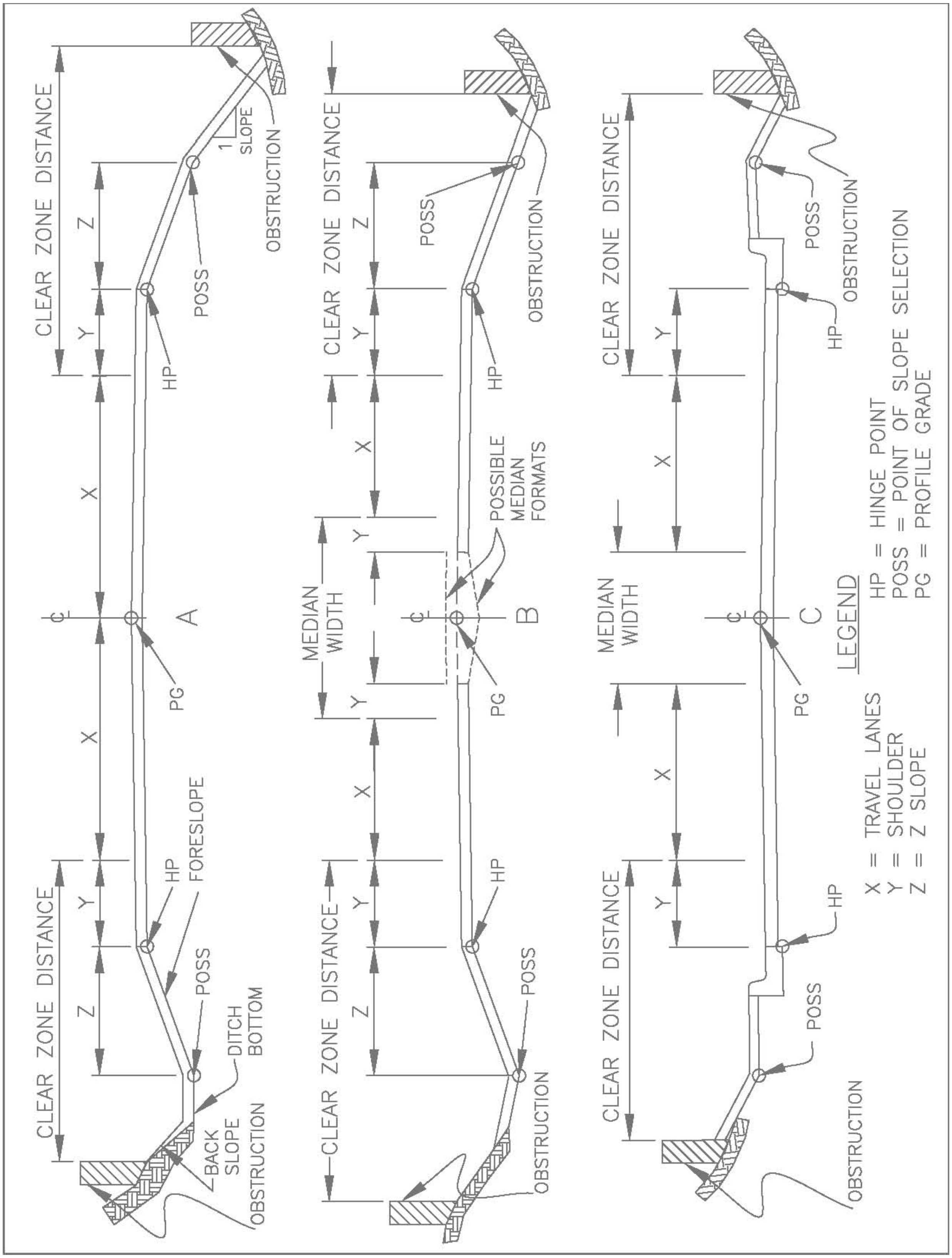


Figure 4-6 General Cross Sectional Information

4.9 TRAFFIC BARRIERS

4.9.1 General Considerations

Traffic barriers are used to minimize the severity of potential accidents involving vehicles leaving the traveled way where the consequences of errant vehicles striking a barrier are less than leaving the roadway. Because barriers are a source of accident potential themselves, their use should be carefully considered. For more detailed information regarding traffic barriers, refer to the *CDOT Safety Selection Guide* **(9)** and the AASHTO *Roadside Design Guide* **(6)**.

Longitudinal barriers are generally denoted as one of three types: flexible, semi-rigid, or rigid. The major difference between the types is the amount of barrier deflection that takes place when the barrier is struck.

Performance level, or barrier capability, lateral deflection characteristics, and the space available to accommodate barrier deflection are important factors in the selection of a longitudinal system. To accommodate the deflection, Guardrail Types 3 and 7 should be placed so that the back of the barrier is at least the minimum distance shown in the *CDOT Standard Plans - M & S Standards* **(2)** from the obstruction.

Consideration should be given to the adaptability of the system to operational transitions, end treatments, and to the initial and future maintenance cost.

Evaluation of the roadside environment entails six options:

- Remove or redesign the obstacle so it can be safely traversed.
- Relocate the obstacle to a point where it is less likely to be struck.
- Reduce impact severity by using an appropriate break-away device.
- Redirect the vehicle by shielding the obstacle with a longitudinal traffic barrier and/or crash cushion.
- Delineate the obstacle if the above alternatives are not appropriate.
- Make no change to existing. See the *CDOT Project Development Manual* **(10)**, Section 2.09.

The sixth option would normally be cost effective only on low-volume and/or low-speed facilities or where engineering studies or safety evaluations show the probability of an accident occurring is low.

Site preparation is an important consideration in the use of traffic barriers. To ensure maximum barrier effectiveness, site conditions should be tailored to the performance characteristics of the particular barrier.

Roadway cross section significantly affects traffic barrier performance. Curbs, dikes, sloped shoulders, and stepped medians can cause errant vehicles to vault or submarine a barrier or to strike a barrier so that the vehicle overturns. Optimum barrier system performance is provided by a relatively level surface in front of the barrier and, for semi-rigid and flexible barriers, beneath and behind the barrier. Where curbs and dikes are used to control drainage, they should be located directly in line with or behind the face of the barrier.

4.9.2 Longitudinal Barriers

Consider the following:

- Height of the barrier affecting site distance; e.g. glare screen or offset barrier affecting sight distance on curves.
- Wildlife being able to clear barriers.
- Maintenance concerns with snow drifting, ease of maintenance, and continuity of type and material.
- Drainage concerns and icing (barrier shadows).
- Adequate room for entrance gating.
- Context-sensitive solutions - see FHWA *Flexibility in Highway Design* **(11)**.
- Materials selection such as wood vs. steel posts and galvanized vs. corrosion-resistant steel.
- Aesthetics – Consider visual impact of selected type.

See the AASHTO *Roadside Design Guide* **(6)**.

4.9.2.1 Roadside Barriers

A roadside barrier is a longitudinal system used to shield motorists from natural or manmade hazards located along either side of a roadway. It may occasionally be used to protect pedestrians, bystanders, and bicyclists from vehicular traffic. Barriers are also used to protect workers in work zones.

Height and slope of the embankment are the basic factors in determining the barrier need through a fill section. The designer should refer to the AASHTO *Roadside Design Guide* **(6)** for determination of barrier needs.

A clear, unobstructed, flat roadside is desirable. The objective of a barrier is to enhance safety. Therefore, a barrier should be installed only if it is clear that the barrier will have lower crash severity potential than the roadside obstacle.

Short lengths of roadside barriers are discouraged. Where needed in two or more closely spaced locations, the barrier should be continuous.

4.9.2.2 Median Barriers

A median barrier is a longitudinal system used to minimize the possibility of an errant vehicle crossing into the path of the traffic traveling in the opposite direction.

Special consideration should be given to barrier needs for medians separating traveled ways at different elevations. The ability of an errant driver leaving the higher elevated roadway to return to the road or to stop diminishes as the difference in elevations increases. The potential for crossover, head-on accidents increases in these situations.

For all divided highways, regardless of median width and traffic volume, the median roadside must also be examined for clear zone hazards.

Barriers should also be considered on outer separations of 50 feet or less adjacent to frontage roads.

Common types of median separation barrier include:

- Double-faced steel W-beam (blocked-out) installed on strong posts.
- Concrete barrier.

Some use is also made of a three or four cable barrier installed on light steel posts, a double- faced steel W-beam installed on weak posts, a double-faced steel thrie beam (blocked-out) installed on strong posts, and a cable-chainlink-fence combination.

During the selection and design of a median barrier, consideration should be given to the possible effect of the barrier on horizontal sight distance.

Precast concrete median barrier can be used for temporary protection of work areas and for guiding traffic during construction.

4.9.3 Bridge Railings

When designing bridge rail, consider protection of pedestrians and cyclists.

The need for traffic barriers generally does not stop at the end of the bridge. The need must be filled by extending the bridge railing with a roadside barrier, which in turn must have a crash-worthy terminal.

At the juncture between a bridge railing and roadside barrier, incompatibility nearly always exists in the stiffness of the two barrier types. This stiffness must be transitioned over a length and with details that will prevent the barrier system from pocketing or snagging an impacting vehicle. For further information, see the *CDOT Standard Plans - M & S Standards* **(2)**.

4.9.4 Crash Cushions

Crash cushions are protective systems that prevent errant vehicles from impacting roadside obstacles by decelerating the vehicle to a safe stop when hit head-on or redirecting it away from the obstacle.

A common application of a crash cushion is at ramp gores where a bridge-rail end exists in the gore. Where site conditions permit, a crash cushion should also be considered as an alternative to a roadside barrier for shielding rigid objects such as bridge piers, overhead sign supports, abutments, and retaining wall ends. Crash cushions may also be used to shield roadside and median barrier terminals.

New highway design should consider alternatives to use of these devices where possible. Where a crash cushion is the best alternative, adequate level space free from curbs or other physical features should be provided. Site preparation is important in using crash cushion design. Site conditions not compatible with the cushion design can compromise cushion effectiveness.

See the CDOT *Safety Selection Guide* **(9)** for guidance in selecting crash cushions.

4.10 MEDIANS

Use of medians will vary according to the type of highway and future developments expected on the highway. Medians may be used to:

- Separate opposing traffic.
- Provide an area for emergency stopping and recovery of errant drivers.
- Allow for left turns and U-turns.
- Provide width for future lanes.
- Minimize headlight glare.
- Provide a refuge area for pedestrians (see Chapter 9).
- Provide area for landscaping.

Median width is measured as the distance between the edges of traveled way and includes inside shoulders. Width of median should be appropriate to its purpose. The primary determinant of required median width is the type of facility. Width may be limited by aesthetic concerns, economics, Right of way limitations, topography, and at-grade intersection signal operations.

Median widths less than 4 feet should be considered separators, not medians. When designing separators, sign width and location should be considered and placement discussed with the Region Traffic Engineer.

Medians may be flush, depressed or raised. Advantages of depressed medians include improved drainage and snow removal. Depressed medians should be sloped downward on a 6:1 slope to a central valley with adequate median drainage provided. Where profile grades differ, engineering judgment must be used to provide a median that will drain properly and be as safe as possible.

Raised medians have application on arterial streets where it is desirable to regulate left-turn movements and control access. Raised medians are typically used in urban settings especially if medians are to be planted. Consider the following: plantings and other landscaping features in median areas may constitute roadside obstacles and may also limit sight distance.

Flush (or painted) medians are often used where two-way left-turn lanes are desired to improve capacity and reduce rear-end accidents. Left-turn lanes may also be placed in the median area. In these cases, the turn lanes are not considered to be part of the median but are designed as a lane.

Normally, the turn lane should be the same width as the travel lanes. Conditions with high truck or bus movement or conditions with limiting geometry may warrant different widths.

4.11 FRONTAGE ROADS

A frontage road is a local auxiliary road located adjacent to a highway. It is primarily used with expressways and freeways although it may be used with any highway.

Among the functions of frontage roads are controlling access, segregating high-speed through traffic from lower speed local traffic, and keeping development in surrounding area from directly affecting the highway.

Specific applications of frontage roads vary with the type of highway. One disadvantage of frontage roads is increased complexity, possibly leading to confusion of drivers.

Frontage road alignment may be parallel or divergent, continuous or broken, one-way or two- way, and on one or both sides of the main highway. Connection of the frontage road and the main highway is one of the more important aspects of frontage road design. Its cross section is dependent on traffic character, volume, and level of service.

From an operational and safety standpoint, one-way frontage roads are preferable to two-way frontage roads. Location of frontage road terminals is dependent on the type of highway it is associated with and the development of the area it serves.

Traffic operations are improved if the frontage roads are located a considerable distance from the main line at the intersecting cross roads in order to lengthen the spacing between successive intersections along the crossroads. In urban areas, a desirable spacing is approximately 150 feet (edge of shoulder to edge of shoulder) between the arterial and the frontage road.

At the intersection, for satisfactory operation with moderate-to-heavy traffic volumes on the frontage roads, the outer separation should be 150 feet or more in width However, wider separations can enhance operations significantly. Outer separations of 300 feet allow for turning movements and provide a minimal amount of vehicle storage.

Narrower separations are acceptable where frontage-road traffic is light, where the frontage road operates one-way only, or where some movements can be prohibited. In some such situations, outer separations as narrow as 8 feet may operate satisfactorily.

Figures 4-7 through 4-10 in the *PGDHS* **(3)** provide schematics of frontage roads.

4.12 OUTER SEPARATIONS

The area between the edge of traveled roadway and edge of traveled way of any street or frontage road is designated as the outer separation. The separation functions as a buffer between highway and local traffic and may be landscaped for improved aesthetics. The width of the outer separation is dependent on the highway classification and the type of street from which it is being separated. Plantings and other landscaping features in outer separators may constitute roadside obstacles. Separations should be designed to prevent unauthorized access between main line and frontage roads.

Type of Frontage Road	**Separation Width**	
	Minimum	***Desirable**
Two-Way Frontage Roads	24 feet	≥ 40 feet
One-Way Frontage Roads	20 feet	≥ 30 feet
Arterial Streets With Frontage Roads	8 feet	
*Use on non-urban highways.		

Table 4-3 Width of Separation for Frontage Roads

Outer separations must meet clear-zone criteria. See the *PGDHS* **(3)** and AASHTO *Roadside Design Guide* **(6)**.

4.13 NOISE CONTROL

4.13.1 General Considerations

Noise is defined as unwanted sound. Motor vehicles generate traffic noise from the motor, aerodynamics, exhaust, and interaction of tires with the roadway. Efforts should be made to minimize the radiation of noise into noise-sensitive areas along the highway. The designer should coordinate with the Region Planning/Environmental Manager to evaluate noise levels and the need for reducing highway traffic noise through location and design considerations.

The physical measurement of human reaction to sound is difficult because there is no instrument that will measure this directly. A close correlation can be obtained by using the A-scale on a standard sound-level meter. The meter yields a direct reading in A-weighted decibels (dBA).

Traffic noise produces varying human reactions. The physical factor of noise is not, in itself, a good predictor of public annoyance; e.g., the reaction is usually less if the noise source is hidden from view. The type of development in an area is another factor that affects the annoyance level. High traffic noise levels are usually more tolerable in industrial areas than in residential areas.

Other factors that influence human reactions to sound are pitch and intermittency. The higher the pitch or the more pronounced the intermittency of the noise, the greater the degree of annoyance.

See also Chapter 18.

4.14 ROADSIDE CONTROL

4.14.1 General Considerations

The efficiency and safety of a highway depend greatly upon the amount and character of roadside interference, most of which originates in vehicle movements to and from business, residences, or other development along the highway. Consult the State Highway Access Code (12) and the Region Access manager for property owner rights of access. Interference resulting from indiscriminate roadside development and uncontrolled driveway connections results in lowered capacity, increased hazards, and early obsolescence of the highway.

See also Chapter 11 and *State Highway Access Code* **(12)** for further information on access.

4.14.2 Driveways

Driveway terminals are, in effect, low-volume intersections; thus, their design and location merit special consideration.

Driveways are directly related to the functional classification of the particular roadway. Where driveways might adversely affect the operation of arterials, the driveways become important links on local streets, where the primary function of the street is to provide access to local establishments.

Driveways should be consolidated whenever possible after consulting with the Region Access Coordinator. An important feature of driveway design is the elimination of large graded or paved areas adjacent to the traveled way upon which drivers can enter and leave the facility at will.

See also Chapter 11 for further information on access.

4.14.3 Mailboxes

Most vehicles stopped at a mailbox will be clear of the traveled way when the mailbox is placed outside an 8-foot wide usable shoulder or turnout.

For guidance on mailbox installations, refer to the latest edition of the AASHTO *A Guide for Erecting Mailboxes on Highways* **(13)**, the AASHTO *Roadside Design Guide* **(6)**, and the *CDOT Standard Plans – M&S Standards* **(2).** Local postal regulations should be consulted for additional criteria.

4.15 TUNNELS

4.15.1 General Considerations

Development of streets or highways may require sections be constructed in tunnels to carry either the streets or highways under or through a natural obstacle, or to minimize the impact of the freeway on the community.

A consultant may be required to design the tunnel, which includes but is not limited to lighting, fire prevention, and electrical and ventilation systems.

4.15.2 Types of Tunnels

Tunnels can be classified into two major categories: tunnels constructed by mining methods, and tunnels constructed by cut-and-cover methods. Of particular interest to the highway designer are the structural requirements of these construction methods and their relative costs.

4.15.3 General Design Considerations

The feeling of confinement in tunnels is unpleasant and traffic noises are magnified. Because tunnels are the most expensive highway structures, they should be made as short as practicable.

Keeping as much of the tunnel length as possible on tangent will not only minimize the length but also improve operating efficiency.

4.15.4 Tunnel Sections

From the standpoint of service to traffic, the design criteria used for tunnels should not differ materially from grade separation structures. The same standards for alignment, profile and for vertical and horizontal clearances generally apply.

Full left- and right-shoulder widths of the approach roadway should be carried through the tunnel. The need for added lateral space is greater in tunnels than under separation structures because of the greater likelihood of vehicles becoming disabled in the longer lengths.

Normally, pedestrians are not permitted in freeway tunnels; however, space should be provided for emergency walking and for access by maintenance personnel. Raised sidewalks, 2.5 feet wide, are desirable beyond the shoulder areas to serve the dual purpose of a safety walk and an obstacle to prevent the overhang of the vehicles from damaging the wall finish or the tunnel lighting fixtures.

4.16 PEDESTRIAN FACILITIES

Appropriate provisions must be made to protect pedestrians from vehicular traffic. Considerations in evaluating the extent of protection required include quantity and variability of pedestrian movement (or activity) and traffic peak and normal volumes, intersection capacity, and site-specific hazards or inconveniences that may influence pedestrian safety.

Suitable treatment may vary from placing a curb or other barrier between the vehicular and pedestrian traffic to construction of a pedestrian overpass or underpass. Pedestrian overpasses and underpasses need to be coordinated with the local agencies and public. When placing pedestrian crossings, care must be exercised to ensure access to persons with disabilities. See *CDOT Standard Plans - M & S Standards* **(2)**, and the requirements of the *Public Rights of Way Accessibility Guidelines (PROWAG)*. For more information contact CDOT's Center for Equal Opportunity.

The project design shall maintain ADA compliant access for and safety of pedestrians, bicyclists, bus stops etc. during construction.

4.16.1 Sidewalks

Sidewalks are most often required in areas with high-pedestrian traffic such as school and commercial areas. Rural areas with such development should be considered for sidewalks along with most urban situations. In urban areas, roadways without shoulders generally should have provision for sidewalks. Sidewalks should be considered for bridges, but the specific sidewalk details may vary.

Sidewalk width may vary due to physical limitations, the presence of a separator between sidewalk and roadway, and the type of development the sidewalk serves. Four to eight foot wide sidewalks are normally used in residential areas and often a 2-foot (minimum) planted strip is provided for maintenance. Where this strip is not present, an additional 2 feet of width is recommended. In commercial areas, the sidewalk typically extends the full width between the roadway and the businesses. Design of sidewalk must include provisions for persons with disabilities; usually those provisions are curb ramps.

The designer should check with local agencies for design impacts.

4.16.2 Sidewalk Curb Ramps

In general, curb ramps within the project limits shall be brought into compliance with the PROWAG (14) compliant CDOT Standard Plan M-608-1 Curb Ramps. Most projects with curb ramps will be required to address ramps that do not meet the minimum requirements for functionally accessibility (as defined in CDOT's Transition Plan). Additional guidance can be found in CDOT's Transition Plan, Chapter 12 of the CDOT Roadway Design Guide and PD 605.1.

Questions about project specific ramps should be directed to the Region ADA Representative.

4.17 BICYCLE FACILITIES

Generally, bicycles can share the roadway with vehicular traffic. In some cases, it is warranted to build separate bikeways. Specific information on warrants and construction requirements for bikeways can be found in Chapter 14.

4.18 BUS TURNOUTS

Bus travel is an increasingly important mode of mass transportation. Bus turnouts serve to remove buses from the traffic lanes. The location and design of turnouts should provide ready access in the safest and most efficient manner possible. Coordinate details with the local transit agency. Inter-Governmental Agreements may be required.

4.18.1 Freeways

The basic design objective for a freeway bus turnout, when exclusive bus roadways are not provided, is that the deceleration, standing, and acceleration of buses are affected on pavement areas clear of and separated from the through-traffic lanes. Speed-change lanes should be long enough to enable the bus to leave and enter the traveled way at approximately the average running speed of the highway without undue discomfort to the passengers. For more details see section 4.19 of the *PGDHS* (**3**).

4.18.2 Arterials

The interference between buses and other traffic can be considerably reduced by providing turnouts clear of the lanes for through traffic.

Coordinate details with the local transit agency.

For more information on bus turnouts, see the AASHTO *Guide for Design of High-Occupancy Vehicle and Public Transportation Facilities* **(15).**

4.18.3 Park-and-Ride Facilities

Park-and-Ride facilities are designed to accommodate:

- Bus loading and unloading.
- Taxis.
- Bicycle parking.
- Parking for bus passengers including persons with disabilities.
- Drop-off facility, plus holding or short-term parking area for passenger pickup.

Coordinate details with local transit agency.

Further details and information can be found in the AASHTO *Guide for Design of High-Occupancy Vehicle and Public Transportation Facilities* **(15)**.

4.19 ON-STREET PARKING

It can generally be stated that on-street parking decreases through-capacity, impedes traffic flow, and increases accident potential. For these reasons, it is desirable to prohibit parking on urban and rural arterial streets. The *PGDHS* **(3)** provides further definition of on-street parking design considerations.

In the design of freeways and control of access-type facilities, as well as on most rural arterials, collectors and local streets, only emergency stopping or parking should be permitted or considered. However, within urban areas and in rural communities located on arterial highway routes, existing and developing land uses necessitate the consideration of on-street parking.

When on-street parking is to be an element of design, parallel parking should be considered. Angle parking presents special problems because of varying vehicle lengths and sight distance problems.

REFERENCES

1. CDOT. *CDOT Pavement Design Manual.* Colorado State Department of Transportation, 2016.

2. CDOT. *CDOT Standard Plans - M & S Standards*. Colorado State Department of Transportation, 2012.

3. AASHTO. *A Policy on Geometric Design of Highways and Streets*, American Association of State Highway and Transportation Officials, Washington, D.C.: 2011.

4. TRB. *Highway Capacity Manual*, Transportation Research Board, Washington D.C.: 2010.

5. AASHTO. *Standard Specifications for Highway Bridges*, American Association of State Highway and Transportation Officials, Washington, D.C.: 2002.

6. AASHTO. *Roadside Design Guide.* American Association of State Highway and Transportation Officials, Washington, D.C.: 2011.

7. CDOT. *CDOT Drainage Design Manual.* Colorado State Department of Transportation, 2017.

8. CDOT. *CDOT Standard Specifications for Road and Bridge Construction*, Colorado Department of Transportation, 2017

9. CDOT. *CDOT Safety Selection Guide.* Colorado State Department of Transportation web site.

10. CDOT *Project Development Manual.* Colorado Department of Transportation, 2013 (with revisions through 2016).

11. FHWA. *Flexibility in Highway Design.* Federal Highway Administration, Washington D.C.: 1997 [https://www.fhwa.dot.gov/environment/publications/flexibility/index.cfm]

12. State Highway Access Code, 2 CCR 601-1, as adopted and amended by the Transportation Commission of Colorado, 1998 (with 2002 revisions).

13. AASHTO. *A Guide for Erecting Mailboxes on Highways*, American Association of State Highway and Transportation Officials, Washington, D.C.: 1994.

14. ADA. *Public Rights of Way Accessibility Guidelines (PROWAG)*, The Access Board, Washington D.C. 2011

15. AASHTO. *Guide for Design of High-Occupancy Vehicle and Public Transportation Facilities.* American Association of State Highway and Transportation Officials, Washington, D.C.: 2004.

CHAPTER 5
LOCAL ROADS AND STREETS

5.0 INTRODUCTION

The following guidelines apply to those roads functionally classified as local roads and local urban streets in accordance with the discussion in Chapter 1 and Chapter 5 of the *PGDHS* **(1)**. In a jurisdictional highway classification, these guidelines apply generally to village or city streets and township and county roads, but not to State Highways.

A local road or residential street primarily serves as access to a farm, residence, business, or other abutting property. Some such roads properly include geometric design and traffic control features more typical of collectors and arterials to encourage the safe movement of through traffic. On these roads, the through traffic is local in nature and extent rather than regional, intrastate, or interstate.

Criteria for use on local roads and streets are presented in the *PGDHS* **(1)**. See Chapter 13 for guidance on low-volume roadways. Other low-volume applications include, but are not limited to, access, forest, recreational, and resource development roads. See Chapter 5 in the *PGDHS* **(1)** for these applications, and check with local agencies for their standards and requirements. Consult Chapter 3 and Chapter 4 of this Guide for details on the basic design elements applicable to this classification of roadway.

5.1 LOCAL RURAL ROADS

5.1.1 General Design Considerations

5.1.1.1 Design Speed

It is necessary for designers to recognize conditions where actual operating speeds typically may exceed the design speed; for example, terrain conditions may limit the overall design speed of a roadway section to a select speed but several long tangents may encourage higher speeds. An older facility may occasionally have a highway curve that has a design speed below the general operating speed of the highway. When this occurs, the common practice is to use an advisory speed sign to warn drivers of the lower safe operating speed on the curve. On new and reconstructed facilities, the curve should be designed to appropriate standards. See Table 5-1, Minimum Design Speeds for Local Rural Roads in the *PGDHS* **(1)** to determine appropriate design speeds as a function of volume and terrain.

5.1.1.2 Grades

Suggested maximum grades for local rural roads are a function of terrain and design speed, see Table 5-2 in the *PGDHS* **(1).**

5.1.1.3 Vertical Curves

Design control for vertical sag and crest curves are provided in Table 5-3 and Table 5-4 of the *PGDHS* **(1).** Criteria for measuring stopping sight distance includes an eye height of 3.5 feet and an object height of 2.0 feet. Passing sight distance includes an eye height of 3.5 feet and the object height is also 3.5 feet.

5.1.2 Intersection Design

Rural intersections may need special design consideration. Accidents may be infrequent, but severity is usually high. Minor improvements can provide major safety benefits. Intersections should avoid steep profile grades and should not be situated just beyond a short-crest vertical curve. Intersections should also be avoided at sharp horizontal curves. Intersections should be designed with adequate corner radii and intersection sight distance. Intersection legs under stop control should intersect at right angles.

5.2 LOCAL URBAN STREETS

5.2.1 General Design Considerations

The design criteria presented in other chapters of this Guide are most applicable to rural and high speed roadways. This section attempts to identify lower design criteria applicable to the lesser functional classes of urban streets that operate at lower speeds.

An urban street is characterized by restricted right of way, stop-and-go traffic, residential, commercial and industrial traffic, pedestrian and bus traffic, bikeways and the special demands and needs these conditions generate. An urban street includes the entire area within the right of way and usually is the product of a comprehensive community development plan. The design values should be those for the ultimately planned development. Typical types of improvements through the urban program include:

- Channelization of intersections.
- Modification of traffic lanes.
- Additional traffic lanes.
- Addition and upgrading of traffic control signs, pavement markings, and signals. Grade separations for pedestrians.
- Major reconstruction or resurfacing.
- New construction.
- Bicycle lanes.

Local agencies desiring to use Federal funds must utilize project designs that meet or exceed these design standards or the minimums also presented in the *PGDHS* **(1)**. The use of Federal funds also requires that the National Environmental Policy Act (NEPA) process be followed because the project design is viewed as a Federal action. Historic districts require special consideration.

The criteria presented in this section are applicable to urban and urbanized area streets with design speeds at or below 40 mph. Local streets have relatively low traffic volumes and the design standards are of a comparatively low order as a matter of practicality.

5.2.2 Design Traffic Volume

The DHV projected to some future design year should be the basis of design. It usually is difficult and costly to modify the geometric design of an existing street unless provision is made at the time of initial construction. Design traffic in these areas should be that estimated for at least 10 years, and preferably 20 years, from the date of completion of construction.

On local residential streets, traffic volume is not usually a major criterion in determining geometric values. Two travel lanes plus additional width for shoulders and parking are usually sufficient.

5.2.3 Design Speed

Design speed is not a major factor for local streets. For consistency in design elements, design speeds ranging from 20 to 30 mph may be used, depending on available right of way, terrain, adjacent development, and other area controls.

In the typical street grid, the closely spaced intersections usually limit vehicular speeds, making the effect of design speed of little significance. Design speeds exceeding 30 mph in residential areas may require longer sight distances and increased curve radii, which would be contrary to the basic function of a local street.

The design speed selected should be a logical choice with respect to the topography, adjacent land use, and type of facility. Once selected, all the pertinent features of the street should be related to that design speed. A street carrying a large volume of traffic may justify a high design speed but a low volume of traffic does not necessarily justify a low design speed. Drivers do not adjust their speeds to the importance of the highway but rather to the physical limitations.

The designer may use the running speed and/or posted speed as logical governing design criteria.

5.2.4 Sight Distance

Minimum stopping sight distance for local streets should range from 115 to 200 feet depending on the design speed (see Table 3-1). Design for passing sight distance seldom is applicable on local streets.

5.2.5 Grades

The grade for residential streets should be as flat as is consistent with the surrounding terrain. When grades are 4 percent or steeper, drainage design may become critical. For streets in industrial areas (with truck traffic) grades should be less than 8 percent and desirably less than 5 percent. See Table 3-4 for maximum grades

To provide for drainage, the minimum preferred grade used for streets with curbs is 0.30 percent but as flat as 0.20 percent may be used when sufficient drainage can be provided. Where bikeways are present, grades may be affected by their separate requirements.

5.2.6 Alignment

Alignment in residential areas should fit closely with the existing topography to minimize the need for cuts or fills. The alignment should not reduce safety but may serve a special purpose if desired by the local planning officials. Street alignment in commercial and industrial areas should be commensurate with the topography but should be as direct as possible. The avoidance or minimizing of involvement with adjacent property associated with hazardous waste or petroleum product contamination may influence the choice of alignment, cross section, and right of way width.

Street curves should be designed with as large a radius as feasible, the minimum radius being 100 feet. Where curves are superelevated, lower values may apply, but the radius should never be less than 75 feet for a 20 mph design speed.

5.2.7 Cross Slope

Pavement cross slope should be adequate to provide proper drainage. Cross slope normally should be as shown in Table 5-1 where there are flush shoulders adjacent to the traveled way.

Surface Type	Range in Cross Slope (%) Local Roads
High	1.5 to 2.0
Intermediate	1.5 to 3.0
Low	2.0 to 6.0
Surface Types	
High = Hard pavements with good retention of properties and support	
Intermediate = Surface treatments to slightly less strict than high type	
Low = Surface treated to loose materials	

Table 5-1 Normal Traveled Way Cross Slopes

The center section of the pavement crown may be parabolic to permit smooth transition of cross slope.

5.2.8 Superelevation

Although superelevation is advantageous for traffic operations, various factors such as wide pavements, abutting properties, drainage, intersections, and access points may make it impractical in built-up areas. Therefore, superelevation is not usually provided on low-speed urban streets in residential and commercial areas. It should be considered in industrial areas or streets where operating speeds are above 40 mph. A maximum superelevation of 0.04 or 0.06 is commonly used. A detailed discussion of superelevation is found in Chapter 3.

5.2.9 Width of Roadway

Street lanes for moving traffic preferably should be at least 10-feet wide. Where feasible, they should be 11-feet wide, and in industrial areas they should be 12-feet wide. Where available or attainable width of right of way imposes severe limitations, 11-foot lanes can be used in industrial areas. Added turning lanes where used at intersections should be 10 to 12 feet wide, depending on the percentage of trucks.

Where local streets carry bicycles, the roadway width should be designed to accommodate the bicycles. Local streets usually provide for two traveled lanes plus parking. Where curb and gutter sections are used, the gutter pan width may be included as a part of the parking lane width. The gutter pan should not be included as part of the travel lane width.

When bicycle facilities are included as part of the design, refer to the AASHTO *Guide for the Development of Bicycle Facilities* **(2)**.

5.2.10 Medians

Medians on low-speed streets are either raised or painted. Local streets rarely have medians and should have justification if the median is a continuous type. Median widths should be designed to accommodate required signing. For the purpose of discussion herein, median areas of 1 to 3 feet in width are considered" separators" or "dividers" and not medians, and may not accommodate required signs.

Openings should be situated only where there is adequate sight distance. The shape and length of the median openings depend on the width of median and the vehicle types to be accommodated. A discussion of the various median types appears in section 4.10.

5.2.11 Drainage

Drainage is an important consideration in urban areas because of high runoff and the flooding potential. Highway drainage facilities are designed to carry water across the right of way and to remove storm water from the roadway itself. These facilities include bridges, culverts, channels, curbs, gutter, and storm sewer systems.

Where drains are available under or near the roadway, the flow is transferred at frequent intervals from the street cross section by grating or curb opening inlets to basins and from there by connectors to drainage channels or underground drains. Properly oriented pedestrian and bicycle-safe inlet grates should be used, where appropriate.

The principal objective in urban drainage design is to control the presence and flow of water on the street surface such that pedestrians, bicyclists, and vehicles are not placed in an unsafe situation.

Economic considerations usually dictate that maximum practical use be made of the street sections for surface drainage. A minimum curb flowline grade for the usual case is 0.30 percent, but a grade of 0.20 percent may be used where there is a high type pavement accurately crowned and supported on firm subgrade.

To provide for proper drainage on local streets, it is desirable to use a minimum crown slope of 2.0 percent (0.02 ft/ft), particularly where the surrounding terrain is relatively flat. This will reduce ponding areas that can contribute to deterioration of pavements and create safety problems. For additional information, see the *CDOT Drainage Design Manual* **(3).**

5.2.12 Cul-De-Sacs and Turnarounds

Check with local agencies for applicable standards and requirements, and see Chapter 5 of the *PGDHS* **(1)**.

5.2.13 Sidewalks

Sidewalks used for pedestrian access to schools, parks, shopping areas, and transit stops and placed along all streets in commercial areas should be provided along both sides of the street. In residential areas, sidewalks are desirable on both sides of the street but need to be provided on at least one side of all local streets. The preferred cross slope is toward the roadway.

Where practical, the sidewalk should be separated from the curb. The three principal reasons for doing this are:

- Greater separation of pedestrians from moving traffic.
- An area for placement of street hardware and traffic signs which will not interfere with pedestrian traffic.
- A location for landscaping.
- A location for placing removed snow.

Maintenance of the area between curb and sidewalk can be difficult and some jurisdictions may desire to eliminate the area in favor of additional sidewalk width. Coordinate with local agencies for maintenance outside of the back of curb.

Clear sidewalk width should be an absolute minimum of 4 feet; 5 feet is desirable. If a continuous sidewalk has a width of 4 feet, a minimum 5-foot by 5-foot passing space needs to be provided at 200-foot intervals. Sidewalk widths of 8 feet or greater may be needed in commercial areas. If roadside appurtenances are situated on the sidewalk adjacent to the curb, additional width is required to secure the clear width.

Pedestrian facilities must be compliant with *PROWAG* **(4)**. Also see Chapter 12 of this Guide.

5.2.14 Sidewalk Curb Ramps

See section 4.16.2.

5.2.15 Border Area

Border area or width is commonly defined as the space from the face of curb to the property line. In many cities, the border width is 10 to 12 feet wide, with at least 1 foot between the outer edge of sidewalk and property line. Border areas should be wider in areas with available right of way or locations where future widening is anticipated.

Consider the following:

- Clear zone
- Street appurtenances
- Sidewalk width
- Utilities
- Landscaping
- Snow storage
- Buffer space between pedestrians and vehicles
- Outer slopes

REFERENCES

1. AASHTO. *A Policy on Geometric Design of Highways and Streets.* American Association of State Highway and Transportation Officials, Washington, D.C.: 2011.
2. AASHTO. *Guide for the Development of Bicycle Facilities*, Washington, D.C.: American Association of State Highway and Transportation Officials, 2012.
3. CDOT. *Drainage Design Manual*. Colorado Department of Transportation, 2017.
4. ADA. *Public Rights of Way Accessibility Guidelines (PROWAG)*, The Access Board, Washington D.C. 2011
5. CDOT. *CDOT Standard Plans – M & S Standards,* Colorado Department of Transportation. 2012

CHAPTER 6
COLLECTOR ROADS AND STREETS

6.0 INTRODUCTION

The following guidelines, applicable to collector roads and streets, are presented on a functional basis. This chapter will be subdivided into rural collectors and urban collectors.

Consult Chapter 3 and Chapter 4 of this Guide for details on the basic design elements applicable to this classification of roadway.

The use of design dimensions more liberal than the minimums described herein is recommended where economically feasible. In all cases, every effort should be made to get the best possible alignment, grade, sight distance, and proper drainage consistent with the terrain, present and proposed development, safety and available funds.

Safety is an important factor in all roadway improvements. On low-volume roads or streets or in urban areas it may not be possible to provide obstacle-free roadsides. As much clear roadside as practical should be provided. The judicious use of flatter slopes, roadside barriers, and warning signs helps to achieve roadside safety. Proper placement of utility features also assists in achieving safer roadsides.

Noise abatement may need to be considered; see section 4.13 and Chapter 18 for more information.

The *Highway Capacity Manual* **(1)** provides the designer with the tools to evaluate the level of service for the highway facility under consideration. Collector streets should generally be designed for 20-year level of service C In heavily developed sections of metropolitan areas, conditions may necessitate the use of level of service D. In rural areas a level of service C is desirable for collector roads. However, level of service D is practical where unusually high traffic volumes exist or where terrain is rolling or mountainous. See section 2.3.1.

Alternate design criteria may be considered for collector roads and streets that carry a lower volume. Consult Chapter 13 for Alternate Standards (Low Volume Roads).

6.1 RURAL COLLECTORS

6.1.1 General Design Considerations

A major part of the rural highway system consists of two-lane collector highways. The rural collector routes generally serve travel of primarily intra-county rather than statewide importance and constitute those routes on which predominant travel distances are shorter than on arterial routes. Rural collectors should be designed to accommodate the highest practical standards compatible with traffic and topography. Basic information necessary for design of collectors includes accident history, traffic volumes, terrain controls, and alignment.

6.1.1.1 Design Speed

Geometric design features should be consistent with a design speed appropriate for the conditions. See Table 6-1, Minimum Design Speeds for Rural Collectors, *PGDHS* **(1)** to determine appropriate design speeds as a function of volume and terrain.

6.1.1.2 Grades

Suggested maximum grades for rural collectors are a function of terrain and design speed, see Table 6-2 in the *PGDHS* **(1).**

6.1.1.3 Vertical Curves

Design control for vertical sag and crest curves are provided in Table 6-3 and Table 6-4 in the *PGDHS* **(1).** Criteria for measuring stopping sight distance includes an eye height of 3.5 feet and an object height of 2.0 feet. Passing sight distance includes an eye height of 3.5 feet and the object height is also 3.5 feet.

6.2 **URBAN COLLECTORS**

6.2.1 General Design Considerations

Urban areas are those places having a population of 5,000 or more within boundaries set by the responsible State and local officials. Urban areas are further subdivided into urbanized areas (population of 50,000 and over) and small urban areas (population between 5,000 and 50,000). For design purposes, the population forecast should be for the design year (usually 20 years).

A collector street is a public facility that includes the entire area within the right of way. The urban collector street also serves pedestrian and bicycle traffic and often accommodates public utility facilities within the right of way. The improvement or development of streets should be based on a functional street classification and should be sensitive to the comprehensive development plan of the local community. The design values should be those for the ultimate planned development. In general, design values for collector streets should be greater than the minimums cited. See Table 6-8, Maximum Grades for Urban Collectors, *PGDHS* **(1)** to determine appropriate grades as a function of design speed and terrain.

Access control on urban collector streets should be used primarily to ensure that access points conform to the adopted criteria for safety, location, design, construction, and maintenance. See Chapter 11 and the *State Highway Access Code* **(2)** for access control requirements.

6.2.2 Parking Lanes

Although on-street parking constitutes a safety problem and impedes traffic flow, parallel parking lanes currently are conventional on many collector streets. Parallel parking will normally be acceptable on urban collectors where sufficient street width is available to provide parking lanes. Where needed in residential areas, a parallel parking lane 8 feet in width should be provided on one or both sides as the conditions of lot size and intensity of development may require. In commercial and industrial areas, parking lane widths should range from 8 to 11 feet. Parking lanes are usually provided on both sides.

Parking lane width determinations should include consideration for likely ultimate use as a lane for moving traffic either during peak hours or continuously.

Where curb and gutter sections are used, the gutter pan width may be included as a part of the minimum width of parking lane, but desirably the lane widths should be in addition to that of the gutter pan.

6.2.3 Drainage

See section 5.2.11.

6.2.4 Sidewalks

See section 5.2.13.

6.2.5 Sidewalk Curb Ramps

See section 5.2.14.

6.2.6 Border Area

See section 5.2.15.

REFERENCES

1. TRB. *Highway Capacity Manual*, Washington, D.C., Transportation Research Board: 2010.
2. Colorado Department of Transportation, Colorado State Transportation Commission, *The State Highway Access Code*, 2 CCR 601-1.

CHAPTER 7
RURAL AND URBAN ARTERIALS

7.0 INTRODUCTION

This chapter provides the general information needed to establish the basis of design for rural and urban arterials.

Consult Chapter 3 and Chapter 4 for details on the basic design elements applicable to this classification of roadway.

7.1 RURAL ARTERIALS

A major part of the rural highway system consists of rural arterials, which range from two-lane roadways to multilane, divided controlled-access arterials.

The appropriate design geometrics for an arterial may be determined from the selected design speed and the design traffic volumes, with consideration of the type of terrain, the general character of the alignment, and the composition of traffic.

7.1.1 General Design Considerations

7.1.1.1 Design Speed

Rural arterials, except for freeways, are normally designed for speeds of 40 to 75 mph depending on terrain, driver expectancy, and whether the design is for new construction or reconstruction of an existing facility. Normal design speeds are shown in Table 3-4.

7.1.1.2 Design Traffic Volume

Check with the Region Traffic Engineer to determine the best source of data for design traffic volumes. The *CDOT Online Transportation Information System (OTIS)* website **(1)** presents base data including Annual Average Daily Traffic (AADT) and design hourly volumes (DHV). Traffic counts may supplement the data. For further information on determining design traffic volumes, see Chapter 2 of the *PGDHS* **(2)**. Base data and additional information such as seasonal variation, directional split, and local-area growth can be obtained from the DTD.

AADT values used to design low-volume rural arterials should be projected to the design year, normally 20 years into the future. DHV is used to design high-volume rural arterials.

7.1.1.3 Levels of Service

Check with the Region Traffic Section for present and future levels of service. Procedures for determining levels of service are presented in the *Highway Capacity Manual* **(3)**. For acceptable degrees of congestion, rural arterials and their auxiliary facilities (i.e., turning lanes, passing sections, weaving sections, intersections, and interchanges) should generally be designed for level of service B except in mountainous areas where level of service C is acceptable.

7.1.1.4 Sight Distance

Sight distance is a direct function of the design speed which greatly influences the level of service on rural arterials. Minimum stopping sight distance must always be provided as a safety requirement. See Chapter 3 for a comprehensive discussion on the subject of sight distance and for tables suitable for the design of arterials.

Ideally, intersections and railroad crossings should be grade-separated or provided with adequate sight distance. Intersections should be placed in sag and/or tangent locations, where practical, to allow maximum visibility of the roadway and pavement markings.

Special consideration should be given to providing adequate decision sight distance at locations such as high-volume intersections and at transitions of roadway widths or numbers of lanes. See Table 3-1.

7.1.1.5 Grades

Table 3-4 gives recommended maximum grades for rural arterials. When vertical curves for stopping sight distance are considered, there are seldom advantages to using the maximum grade values except when grades are long. Grades below the maximum are always desirable, but a grade of 0.5 percent should be considered the minimum.

7.1.1.6 Number of Lanes

The number of lanes required is determined by volume, level of service, and capacity conditions. A multilane arterial refers to four or more lanes. The required number of lanes is determined by procedures in the *Highway Capacity Manual* **(3)**.

7.1.1.7 Superelevation

When the use of curves is required on a rural arterial alignment, a superelevation rate compatible with the design speed must be used. Adjustments in design runoff lengths may be necessary for smooth riding, drainage, and appearance. See the *CDOT Standard Plans - M & S Standards* **(4)** for superelevation rates. Chapter 3 provides a detailed explanation of superelevation rates and runoff lengths for design speed.

7.1.1.8 Cross Slope

See section 4.1.2.

7.1.1.9 Vertical Clearances

New or reconstructed roadway structures shall provide a minimum of 16.5 feet of clearance over the entire roadway width, which includes 6 inches for future resurfacing of the underpassing road. Existing structures that provide 14 feet of clearance, if allowed by local statute, may be retained. In highly urbanized areas, a minimum clearance of 14 feet may be allowed only if there is an alternate route with 16 feet of clearance. Additional information on vertical clearances, including non-roadway vertical clearances, is given in Table 3-3.

7.1.1.10 Structures

Bridges to remain in place shall have adequate strength and be at least the width of the traveled way plus 2 feet of clearance on each side, but should be considered for ultimate widening or replacement.

Consult with Staff Bridge on all new and existing structures.

7.1.1.11 Widths

Table 7-1 provides values for the width of traveled way and shoulder that should be considered for the volumes indicated. Shoulders should be usable at all times regardless of weather conditions. On high-volume highways the shoulders shall be paved. The shoulder should be constructed to a uniform width for relatively long stretches of roadway. For additional information regarding shoulders, refer to section 4.3.

Design Speed (mph)	**Minimum width of traveled way (ft) [a] for specified design volume (veh/day)**			
	Under 400	400 to 1500	1500 to 2000	Over 2000
40	22	22	22	24
45	22	22	22	24
50	22	22	24	24
55	22	22	24	24
60	24	24	24	24
65	24	24	24	24
70	24	24	24	24
75	24	24	24	24
All speeds	**Width of shoulder (ft) [b]**			
	4	6	6	8

[a] On roadways to be reconstructed, an existing 22-ft traveled way may be retained where alignment and safety records are satisfactory.

[b] Shoulders on arterials shall be paved; however, where volumes are low or a narrow section is needed to reduce construction impacts, the paved shoulder may be reduced to 2 ft.

Table 7-1 Minimum Width of Traveled Way and Shoulder

7.1.1.12 Clear Zones

When fixed objects or non-traversable slopes fall within the roadside clear zones discussed in Chapter 4.6.1 of the *PGDHS* **(2)**, refer to the AASHTO *Roadside Design Guide* **(5)** for guidance in selecting the appropriate treatment. For guardrail installations, see *CDOT Standard Plans - M & S Standards* **(4)**.

7.1.1.13 Right of Way

A uniform width of right of way may be convenient, but there are special cases where additional right of way may be desirable. These cases include locations where the side slopes extend beyond the normal right of way, where greater sight distance is desirable, at intersections and junctions with highways, at-grade railroad crossings, and for environmental considerations.

Consider the following when determining right of way widths:

- Local conditions such as drainage and snow storage.
- Rounding of the slopes.
- Extending right of way 10 to 15 feet from the bottom of the toe or the top of the cut for level terrain, 15 to 20 feet for mountainous terrain.
- Utility corridor or easement.
- Irrigation features.
- Future capacity improvements.
- Transit alternatives.
- Additional needs for maintenance and utility purposes.

On staged construction, it may be desirable to construct the initial two lanes off-center so that the future construction will not interfere with the traffic or waste the investment in the initial grading and surfacing stage.

See Table 4-1.

7.1.1.14 Ultimate Development of Four-Lane Divided Arterials

Where it is anticipated that a DHV for the design year will be in excess of the design capacity of the two-lane arterial, the initial improvement should be patterned to the ultimate development of a four-lane divided arterial and provisions made for acquisition of the necessary right of way.

Even where right of way is restricted, some form of separator should be used in the ultimate facility, with a median at least 4-feet wide, but preferably wider.

In the ultimate development of a four-lane divided arterial, the initial two-lane surfacing should be constructed to form one of the two-lane one-way surfaces.

7.1.2 Multilane Undivided Arterials

The minimum required sight distance at all points is the stopping sight distance, because passing is accomplished without the necessity of using an opposing traffic lane. Longer than minimum required stopping sight distance is desirable, as it is on any type of arterial.

Adequate shoulders, which encourage drivers to use them in emergencies, are essential on multilane undivided arterials.

If traffic volumes require the construction of multilane arterials in rural areas where speeds are apt to be high, it is generally considered that opposing traffic should be separated by a depressed median or barrier. All arterials on new locations requiring four or more lanes should be divided. Improvement of an existing two-lane arterial to a multilane facility should include a depressed median or barrier.

Undivided arterials with four or more lanes are most applicable in urban and suburban areas where there is concentrated development of adjacent land.

7.1.3 Divided Arterials

7.1.3.1 General Features

A divided arterial is one with separated lanes for traffic in opposite directions. It may be situated on a single roadbed or two widely separated roadways. The width of the median may vary and is governed largely by the type of area, character of terrain, intersection treatment, and economics. An arterial is not normally considered to be divided unless two full lanes are provided in each direction of travel and the median is 4 feet or wider and constructed or marked in a manner to preclude its use by moving vehicles except in emergencies or for left turns. A four-lane rural facility should have adequate median width to provide for protected left turns.

The principal advantages of dividing the multilane arterial are increased safety, comfort, and ease of operation. Of significance is the reduction of head-on collisions and virtual elimination of such accidents on sections with wide medians or with a median barrier. Pedestrians crossing the divided arterial are required to watch traffic in only one direction at a time and are provided a refuge at the median, particularly if a raised island is provided. Where the median is wide enough, crossing and left-turning vehicles can slow down or stop between the one-way pavements to take advantage of breaks in traffic and cross when it is safe to do so.

7.1.3.2 Lane Widths, Cross Slope, and Shoulders

See Chapter 4.

7.1.3.3 Medians

On highways without at-grade intersections, the median may be as narrow as 4 to 6 feet under restricted conditions but wider medians should be provided wherever feasible. A wide median allows the use of independent profiles. Median widths of more than 60 feet are undesirable at intersections that are signalized or may need to be signalized in the future.

Medians should be designed for the appropriate design vehicle such as buses.

While medians as narrow as 4 to 6 feet may be required under restricted conditions, medians 12 to 30 feet wide provide protection for left-turning vehicles at intersections. If urban situations are expected in the future, consider a median of 18 feet which accommodates a 12-foot turn lane and 6 feet for curb, gutter, and signage.

Median widths from 30 to 50 feet should be carefully considered from an operational standpoint at intersections. These widths do not provide median storage space for larger vehicles crossing the median. Also, these widths may encourage the driver to attempt the crossing independently leaving a portion of the vehicle unprotected from through traffic. These widths, even with these problems, normally operate quite well and apparently are within the realm of normal operational expectations of the driver.

For left-turn design, refer to Urban Arterials in Chapter 7.3 of the *PGDHS* **(2)**. See Figure 7-4 of the *PGDHS* **(2)** for typical medians on divided arterials. See also Chapter 4.10.

7.1.3.4 Climbing Lanes on Multilane Arterials

Climbing lanes generally are not as easily justified on multilane arterials as on two-lane arterials. A full discussion on the need for climbing lanes and their derivation is found in section 3.3.5.

7.1.4 Access Management

Consult the *State Highway Access Code* **(6)**, and Chapter 11 of this Guide.

7.2 URBAN ARTERIALS

7.2.1 General Considerations

Urban arterials carry large traffic volumes within and through urban areas. Their design varies from freeways with fully controlled access to two-lane streets.

The urban arterial system, which includes arterial streets and freeways, serves the major centers of activity of a metropolitan area, the highest traffic volume corridors, and the longest trips.

7.2.1.1 Design Speed

Design speeds for urban arterials generally range from 30 to 60 mph. Lower speeds apply in central business districts and in more developed areas, while higher speeds are more applicable to outlying suburban and developing areas.

7.2.1.2 Design Traffic Volume

See 7.1.1.2.

7.2.1.3 Levels of Service

Check with the Region Traffic Section for present and future levels of service. Procedures for determining levels of service are presented in the *Highway Capacity Manual* **(3)**. For acceptable degrees of congestion, rural and suburban arterials and their auxiliary facilities, i.e., turning lanes, weaving sections, intersections, interchanges, and traffic control systems (traffic signals, etc.), should generally be designed for level-of-service C for the particular design year. Heavily developed sections of metropolitan areas may necessitate the use of level of service D for the particular design year. When level-of-service D is selected, it may be desirable to consider the use of one-way streets or alternative bypass routes to improve the level of service.

7.2.1.4 Sight Distance

The sight distance values given in Table 3-1 are also applicable to urban arterial design.

7.2.1.5 Grades

The grades selected for an urban arterial may have a significant effect on its operational characteristics. For example, steep grades affect truck speeds and overall capacity. On arterials having large numbers of trucks and operating near capacity, flat grades should be considered to avoid undesirable reductions in speed. Steep grades also result in operational problems at intersections, particularly during adverse weather conditions. For these reasons, it is desirable to provide the flattest practicable grades while providing minimum gradients as required to ensure adequate longitudinal drainage in curbed sections. See Table 3-4.

7.2.1.6 Vertical Clearances

See section 7.1.1.9.

7.2.1.7 Curbs and Shoulders

Shoulders are desirable on any highway, and urban arterials are no exception. Shoulders contribute to safety by affording maneuver room and providing space for immobilized vehicles. They offer a measure of safety to the occasional pedestrian in sparsely developed areas where sidewalks are not appropriate. Shoulders serve as speed-change lanes for vehicles turning into driveways and provide storage space for plowed snow.

Despite the many advantages of shoulders on arterial streets, their use is generally limited by restricted right of way and the necessity of using the available right of way for traffic lanes. A raised curb at the outer edge of the shoulder is usually necessary in heavily developed areas as a means of controlling access and preventing deterioration of the shoulder. These requirements usually result in a cross section having a uniform pavement design with vertical-type curbs.

See Chapters 4 and 14 of this Guide and Chapter 4 of the *PGDHS* **(2)**.

7.2.1.8 Number of Lanes

A capacity analysis should be performed to determine the number of lanes.

7.2.1.9 Width of Roadway

Roadway width should be adequate to accommodate the traffic lanes, medians, curbs, and the required clearances from barrier faces. Parking on an arterial street should be considered only when provision is required because of existing conditions.

Lane widths may vary from 11 to 12 feet. Eleven-foot lanes are used quite extensively for urban arterial street designs. Twelve-foot lane widths are most desirable and are generally used on all higher speed, free-flowing, principal arterials. Under interrupted flow operating conditions at low speeds up through 40 mph, narrower lanes may be adequate.

The use of minimum requirements should be avoided if possible.

7.2.1.10 Geometric Design Type

Geometric design type wi1l be either Type B, Type A, or Type AA based on the number of lanes required for the facility. See Table 4-1.

7.2.1.11 Medians

Medians are a desirable feature of arterial streets and should be provided where space permits. Design of medians and median barriers is discussed in Chapter 4 of this Guide and the *PGDHS* **(2)**. Additional information on medians relative to urban arterials is given in Chapter 7 of the *PGDHS* **(2).**

7.2.1.11.1 Median Considerations

The designer should consider the possible future developments affecting the highway when selecting a median design. Two-lane highways, particularly in urban areas, may eventually require widening to four lanes. The ultimate use of the highway influences the selection of design criteria such as design speed, access openings, and whether rural or urban. Median design should be compatible with the eventual functional category of the highway.

Consider the following:

- Need for additional lanes in the future.
- Need for turning/storage lanes.
- Signalization.
- Improvements along outside lane (right turn provisions, curb and gutter, etc.).
- Need for parking.
- Pedestrian/bikeway requirements.
- Access requirements or controls.
- Clear zone requirements.

The above parameters compete with each other for available space and right of way. Even where right of way is restricted, some form of median should be provided on four-lane highways. The use of minimum requirements should be avoided if possible.

7.2.1.11.2 Width of Median

For rural and urban arterials:

- Four to 6-foot medians may be used under very restricted conditions.
- Twelve to 30-foot medians are desirable to provide protection for left turn lanes.
- Four to 8-foot medians should be avoided where left turns are common.

The width of median is the distance between inside edges of traveled ways. This width is dependent upon the type of facility, topography, and right of way considerations. For curbed medians, the gutter is not included in the traveled way, and the distance from face-to-face of curb should not be less than 2 feet.

Desirably, the median should be at least 18 feet wide for a 12-foot median turn lane and a 6-foot median separator. At restricted locations, a 10-foot lane with a 2-foot painted median separator may be used. The median width to accommodate left turning movements should be at least 12 feet. A median only 4 feet wide is better than none; however, each additional foot provides an added increment of safety and improved operation. Median widths less than 4 feet should be painted. Reasons for using distances less than those stated above shall be documented in a design decision letter.

At some locations, it may be necessary to forego the curbed median altogether and provide only pavement striping; e.g., a continuous two-way left turn lane. In these cases, the lane is not considered a median and should be designed as a separate lane (11 to 16 feet wide).

7.2.1.11.3 Width of Median Lanes

At commercial or business locations and where truck turning movements are high, the median turn lane should be 12 feet wide, and in no case less than the main line width. Turning movements with 5 percent or more of trucks should be studied for use of the 12-foot requirements. For a truck count of 10 percent or greater, it is recommended that 12-foot lanes be used. Buses should be treated in the same way as trucks.

When trucks and buses are not a consideration, auxiliary lanes may be reduced to a minimum of 10 feet if warranted. Conditions that may warrant the lesser width are:

- Narrow right of way.
- Narrow street widths.
- Narrow existing medians being modified at intersections.
- Safety improvements that furnish the minimum rather than nothing at all.

It is not desirable to reduce median lane widths to the minimum on any type of arterial street or on the higher speed rural roads.

Auxiliary lane design varies between urban and rural applications. On urban streets, the lower speeds, driver expectancy of sudden movements and presence of pedestrians, traffic signals, lighting, and narrow right of way help to justify accepting lower standards (use of minimums), if needed. On rural roads, the driver is accustomed to higher speeds, and median design should meet the desirable standards. Auxiliary lanes desirably should have the same width as provided on the main travel lanes. On arterial highways this width would normally be 12 feet.

7.2.1.11.4 Cross Slope for Curbed Medians

Curbed medians normally should be crowned with a slope of 4 percent for self-cleaning and drainage.

7.2.1.11.5 Median Contrast

Pavement and median areas should contrast effectively in color, texture, or both, under wet and dry conditions. Surface types that contrast with the traveled way should be used for surfacing medians.

7.2.1.11.6 Median Configuration and Typical Design

The type of treatment used in a median configuration is usually dependent on local practice and available right of way widths. The type selected should always be compatible with drainage and required street appurtenances. It is desirable that the median be of uniform width; however, where intersections are widely spaced (e.g., 0.5 miles or more), the width of median may be varied. This can be accomplished by using a narrow width between intersections where necessary for economy and gradually widening on the approach to the intersection to accommodate the left-turn lane. Where possible, transitions between varying median widths should be made on curves in order to avoid reverse curvature.

7.2.1.12 Drainage

Inlets that are safe for bicycles should be located adjacent to and upstream of intersections, pedestrian facilities, and at intermediate locations where necessary. Additional inlets should be provided in sag locations to avoid ponding of water where the grade flattens to zero percent and to mitigate flooding should an inlet become clogged.

7.2.1.13 Parking Lanes

See Chapter 7.3.3 of the *PGDHS* **(2)**.

7.2.1.14 Borders and Sidewalks

Coordinate closely with the local agency and ensure ADA compliance. See sections 4.16, 5.3.13-15, and 6.2.4-6 for a more thorough discussion.

7.2.1.15 Roadway Width for Bridges

See Chapter 4, Table 4-1.

7.2.1.16 Clear Zones

AASHTO standards for clear zone design are recommended for urban arterials whenever feasible. On curbed street sections, this design is often impracticable, particularly in restricted areas. In those areas, a clearance from curb face to object of 1.5 feet or wider where possible should be the minimum. A 3-foot clearance will desirably be provided particularly near turning radii at intersections and driveways. This desirable offset provides the clearance required for overhang of trucks from striking the object. When pedestrians are not a factor, obstructions should be well set back, protected, or provided with breakaway features. Guardrail may be considered in special cases. Refer to the AASHTO *Roadside Design Guide* **(5)**.

7.2.1.17 Right of Way Width

The right of way should be wide enough to accommodate all of the cross-sectional elements throughout the project. This usually precludes a uniform right of way width since there are typically many situations where additional width is desirable. Such situations occur where the side slopes extend beyond the normal right of way, for clear areas at the bottom of traversable slopes, for wide clear areas on the outside of curves, where greater sight distance is desirable, at intersections and junctions with highways, at railroad-highway grade crossings, for environmental considerations, and for maintenance access.

Local conditions such as drainage and snow storage should be considered in determining right of way widths. Where additional lanes may be needed in the future, the initial right of way width should be adequate to provide the wider roadway section.

7.2.1.18 Intersection Design

Chapter 9 discusses intersection development in detail. It is recommended that each individual intersection be carefully evaluated in the early design phases.

7.2.1.19 Lighting

See section 3.7.

REFERENCES

1. CDOT. *CDOT Online Transportation Information System (OTIS)*. [http://dtdapps.coloradodot.info/otis]

2. AASHTO. *A Policy on Geometric Design of Highways and Streets*, American Association of State Highway and Transportation Officials (AASHTO), Washington, D.C.: 2011.

3. TRB. *Highway Capacity Manual*, Transportation Research Board, Washington, D.C., 2010.

4. CDOT. *Standard Plans - M & S Standards*, Colorado State Department of Transportation, 2012.

5. AASHTO. *Roadside Design Guide*, American Association of State Highway and Transportation Officials, Washington. D.C.: 2011.

6. CDOT. *State Highway Access Code*, 2 CCR 601-1, as adopted and amended by the Transportation Commission of Colorado, 1998 (with 2002 revisions).

CHAPTER 8
FREEWAYS

8.0 INTRODUCTION

The discussion in this chapter on freeways applies to both urban and rural freeways, except where noted. Supplemental requirements for the design of Interstate highways can be found in AASHTO's companion booklet, A Policy on Design Standards – Interstate System (1).

The highest type of arterial highway is the freeway, which is defined as an arterial highway with full control of access and no at grade crossings or connections. Full control of access is the condition where the right of owners or occupants of abutting land to access a freeway is fully controlled by public authority. Access connections to the freeway are with selected public roads only. Crossings at grade or direct private driveway connections are prohibited.

Essential freeway elements include: medians; grade separations; ramps; and in some cases, frontage roads. This chapter identifies the various types of freeways, emphasizes selected features, and discusses other design details unique to these freeway types.

Consult Chapters 3 and 4 for details on the basic design elements applicable to this classification of roadway.

8.1 GENERAL DESIGN CONSIDERATIONS

The following discussions are for both urban and rural freeways, except as noted.

8.1.1 Design Speed

Design speed should be consistent with the anticipated operating speed and driver expectations.

The design speed of urban freeways should not be less than 50 mph. On many urban freeways, particularly in developing areas, a design speed of 60 mph can be provided with little additional cost.

A design speed of at least 70 mph is typical on rural freeways; however, when practical, a design speed of 75 mph or higher should be considered on rural freeways where the speed limit is currently, or could become, 75 mph. Where terrain is mountainous, a design speed of 50 to 60 mph may be used.

8.1.2 Design Traffic Volumes

Both urban and rural freeways, especially in the case of new construction, are normally designed to accommodate traffic projections for a 20-year period. Some elements of freeway reconstruction may be based on a lesser design period. Specific capacity requirements should be determined from directional design hourly volumes (DDHV) for the appropriate design period.

8.1.3 Levels of Service

Freeways and their auxiliary facilities, i.e., ramps, mainline weaving sections, should generally be designed for level-of-service C. In heavily developed sections of metropolitan areas, conditions may necessitate the use of level-of-service D. In rural areas, level-of-service B is desirable for through and auxiliary lanes, although level-of-service C may be acceptable on auxiliary facilities carrying unusually high volumes of traffic. Designers should strive to provide the highest level of service practical and consistent with anticipated conditions. Procedures for traffic operational analyses for freeways are found in the *Highway Capacity Manual* **(2)**.

8.1.4 Pavement and Shoulders

Freeways have a minimum of two through-traffic lanes for each direction of travel. Through-traffic lanes should be 12 feet wide. Cross slopes should be 2 percent. There should be continuous paved shoulders on both the right and left sides of all freeway facilities. The usable paved width of the right shoulder should be at least 10 feet. On four-lane freeways, the median (left) shoulder is normally 4 feet wide. See section 4.3.

On freeways of six or more lanes, the usable paved width of the median shoulder should also be 10 feet. Where the DDHV for truck traffic exceeds 250 veh/h, a paved shoulder width of 12 feet should be considered.

8.1.5 Curbs

In general, neither vertical nor sloping type curbs are desirable for use on high-speed roadways. For more information, refer to the discussion on curb types and their placement in section 4.6 of this guide, the *PGDHS* **(3)** and the *AASHTO Roadside Design Guide* **(4)**.

8.1.6 Superelevation

See section 3.2.3.

8.1.7 Grades

Maximum grades as a function of design speed and type of terrain are given in Table 3-4. Where sustained upgrades are required, the need for climbing lanes should be investigated.

8.1.8 Structures

The design of bridges, culverts, walls, tunnels, and other structures shall be in accordance with the principals of the current AASHTO *Standard Specifications for Highway Bridges* **(5).** Structures carrying freeway traffic should provide an HS20-44 design loading.

The clear width on bridges carrying freeway traffic should be as wide as the approach pavement and paved shoulders (see Chapter 15 for shoulders less than 6 feet wide). Structures carrying ramps should provide a clear width equal to the ramp width and paved shoulders.

8.1.9 Vertical Clearance

See section 3.3 and Table 3-3.

8.1.10 Horizontal Clearance to Obstructions

Freeways should have clear zone widths consistent with their operating speed and side slopes as discussed in section 4.5 of this Guide, in Chapter 8 of the *PGDHS* **(3)**, and in the *AASHTO Roadside Design Guide* **(4)**.

8.1.11 Outer Separations, Borders, and Frontage Roads

See section 4.12.

8.2 RURAL FREEWAYS

8.2.1 Alignment and Profile

Rural freeways should have smooth-flowing horizontal and vertical alignments. Proper combinations of flat curvature, shorter tangents, gentle grades, variable median widths, and separate roadway elevation enhance the safety and aesthetic aspect of freeways. Changing median widths on tangent alignments should be avoided where practical to avoid a distorted appearance.

Rural freeways can usually be constructed near ground level with smooth and relatively flat profiles. The profile of a rural freeway is controlled more by drainage and earthwork considerations and less by the need for frequent grade separations and interchanges.

8.2.2 Medians

Median widths of about 50 to 100 feet are common on rural freeways. A 50-foot median provides for 6-foot graded shoulders and 6:1 foreslopes with a 3-foot median ditch depth and adequate space for vehicle recovery. A 100-foot median would permit the designer to use independent profiles in rolling terrain to blend the freeway more appropriately with the environment while maintaining flat slopes for vehicle recovery. In flat terrain, the 100-foot median is also suitable when stage construction includes the addition of two future 12-foot traffic lanes.

Where the terrain is extremely rolling or the land is not suitable for cultivation or grazing, a wide variable median having an average width of 150 feet or more may be attainable. This permits the use of independent roadway alignment, both horizontally and vertically, to the best advantage in blending the freeway into the natural topography. Foreslopes and backslopes used within the clear zone should provide for vehicle recovery. The remaining median width may be left in its natural state of vegetation, trees, and rock outcroppings.

In areas where right of way restrictions dictate or in mountainous terrain, median widths in the range of 10 to 30 feet may be necessary. In such instances, the median is usually paved. Due to the usual developing-area traffic volumes as well as operational characteristics in mountainous areas, a median barrier is usually warranted as a safety measure.

Emergency crossovers on rural freeways are normally provided where interchange spacing exceeds 5 miles. Between interchanges, emergency crossovers are spaced at 3 to 4-mile intervals. Maintenance crossovers may be required at one or both ends of interchange facilities for the purpose of snow removal and at other locations to facilitate maintenance operations. Crossovers should not be located closer than 1,500 feet to the end of a speed-change taper of a ramp or to any structure. Crossovers should be located only where above-minimum stopping sight distance is provided and preferably should not be located within curves requiring superelevation.

The width of the crossover should be sufficient to provide safe turning movements and should have a surface capable of supporting the maintenance equipment used on it. Crossovers should not be placed in restricted-width medians unless the median width is sufficient to accommodate a vehicle length of 25 feet or more.

Where median barriers are employed, each end of the barrier at the opening requires a crashworthy terminal.

For further information on medians, refer to section 4.10 of this Guide, the *PGDHS* **(3)**, and the AASHTO *Roadside Design Guide* **(4)**.

8.2.3 Sideslopes

Flat, rounded sideslopes, fitting with the topography and consistent with available right of way, should be provided on rural freeways. Foreslopes of 6:1 or flatter are recommended in cut sections and for fills of moderate height. Where fill heights are intermediate, a combination of recoverable and non-recoverable slopes may be used to provide the acceptable vehicle recovery area. For high fills, steeper slopes protected by guardrail may be needed. In addition, backslopes of 3:1 or flatter permit landscaping and erosion control practices and ease of maintenance operations.

8.2.4 Frontage Roads

See section 4.11 of this Guide and the *PGDHS* **(3)**.

8.3 URBAN FREEWAYS

8.3.1 Medians

The median on urban freeways should be as wide and flat as feasible. Extra median width also can be used for transit or to provide additional lanes if more capacity is needed in the future. The minimum median width for a four-lane urban freeway is 10 feet, which provides for two 4-foot shoulders and a 2-foot median barrier. For freeways with six or more lanes, the minimum width is 22 feet, and preferably 26 feet, when truck traffic exceeds 250 DDHV, to accommodate the wider median shoulder. For these minimum median widths, a median barrier is always required. Additional horizontal clearance may be required to provide minimum stopping sight distance along the inside lane on sharper curves.

Median crossovers for emergency or maintenance purposes are generally not warranted on urban freeways due to the close spacing of interchange facilities and the extensive development of the abutting street network.

8.3.2 Depressed Freeways

Information for the following is found in Chapter 8 of the *PGDHS* **(3)**:

- Slopes and walls
- Typical cross section
- Restricted cross section
- Walled cross section

8.3.3 Elevated Freeways

Information for the following is found in Chapter 8 of the *PGDHS* **(3)**:

- Medians
- Ramps and terminals
- Frontage roads
- Clearance to building lines
- Typical cross section
- Viaduct freeways without ramps
- Two-way viaduct freeways with ramps
- Freeways on earth embankments

8.3.4 Ground-Level Freeways

Information for the following is found in Chapter 8 of the *PGDHS* **(3)**:

- Typical cross sections
- Restricted cross sections

8.3.5 Combination-Type Freeways

Information for the following is found in Chapter 8 of the *PGDHS* **(3)**:

- Profile control
- Cross-section control

8.3.6 Special Freeway Designs

Information for the following is found in Chapter 8 of the *PGDHS* **(3)**:

- Reverse-flow roadways
- Dual-divided freeways
- Freeways with collector-distributor roads

8.3.7 Accommodation of Transit and High-Occupancy Vehicle Facilities

Information for the following is found in Chapter 8 of the *PGDHS* **(3)**:

- Buses
- Rail transit

REFERENCES

1. AASHTO. *A Policy on Design Standards – Interstate System*, American Association of State Highway and Transportation Officials Washington, D.C.: 2016.
2. TRB. *Highway Capacity Manual*, Transportation Research Board, Washington, D.C.: 2010.
3. AASHTO. *A Policy on Geometric Design of Highways and Streets*, American Association of State Highway and Transportation Officials, Washington, D.C.: 2011.
4. AASHTO. *Roadside Design Guide*, American Association of State Highway and Transportation Officials, Washington, D.C.: 2011.
5. AASHTO. *AASHTO Standard Specifications for Highway Bridges*, American Association of State Highway and Transportation Officials, Washington, D.C.: 2002

CHAPTER 9
INTERSECTIONS

9.0 INTRODUCTION

Intersections are intended to operate with vehicles, pedestrians, and bicycles proceeding in many directions, often at the same time. At such locations, traffic movements on two or more facilities are required to occupy a common area. It is this unique characteristic of intersections, the repeated occurrence of conflicts, that is the basis for most intersection design standards, criteria, and proper operating procedures.

An intersection is defined as the general area where two or more highways join or cross, including the roadway and roadside facilities for traffic movements within it. Each highway radiating from an intersection and forming part of it is an intersection leg. The common intersection of two highways crossing each other has four legs. It is not recommended that an intersection have more than four legs.

An intersection is an important part of a highway system because, to a great extent, the efficiency, safety, speed, cost of operation, and capacity depend on its design. Each intersection involves through or cross-traffic movements on one or more of the highways concerned and may involve turning movements between these highways. These movements may be handled by various means, such as signals, signing, and channelization, depending on the type of intersection.

9.1 GENERAL DESIGN CONSIDERATIONS AND OBJECTIVES

The main objective of intersection design is to provide convenience, ease of use, and comfort to the people traversing the intersection while facilitating the efficient movement of passenger cars, buses, trucks, bicycles, and pedestrians. The design should be fitted closely to the natural transitional paths and operating characteristics of the users.

Four basic elements enter into design considerations of intersections:

- Human factors
- Traffic considerations
- Physical elements
- Economic factors

Although intersections have many common factors, they are not subject to class treatment, and they must be treated as individual problems.

9.1.1 Intersection Functional Area

Intersections are defined by both the functional and physical areas as shown in Figure 9-1. The physical intersection area traditionally extends from point of return perpendicular across the roadway. The functional area of an intersection extends both upstream and downstream of the physical intersection area including speed change lanes and their tapers.

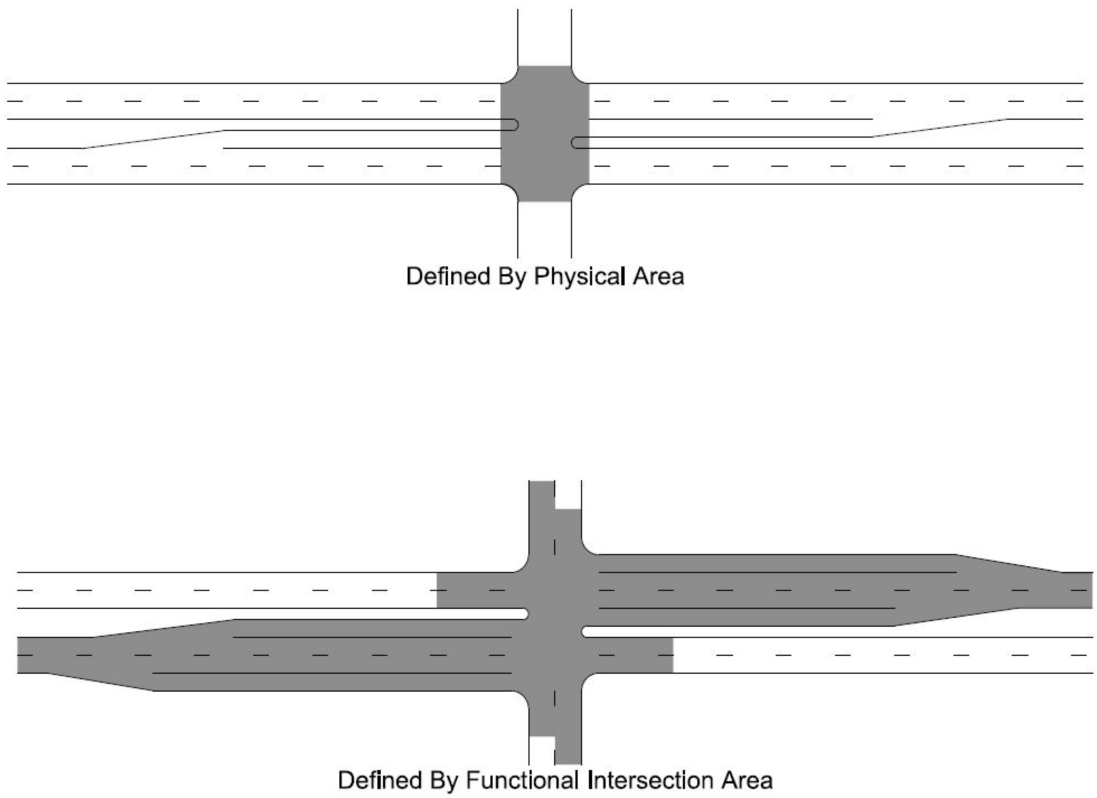


Figure 9-1 Physical and Functional Intersection Area

The functional area of the approach to an intersection or access point consists of perception-reaction decision distance, maneuver distance, and storage distance. Elements of the functional area are displayed below in Figure 9-2. The maneuver distance includes the length needed for breaking and lane changing, if lane changing is not provided, the distance involves breaking to a comfortable stop.

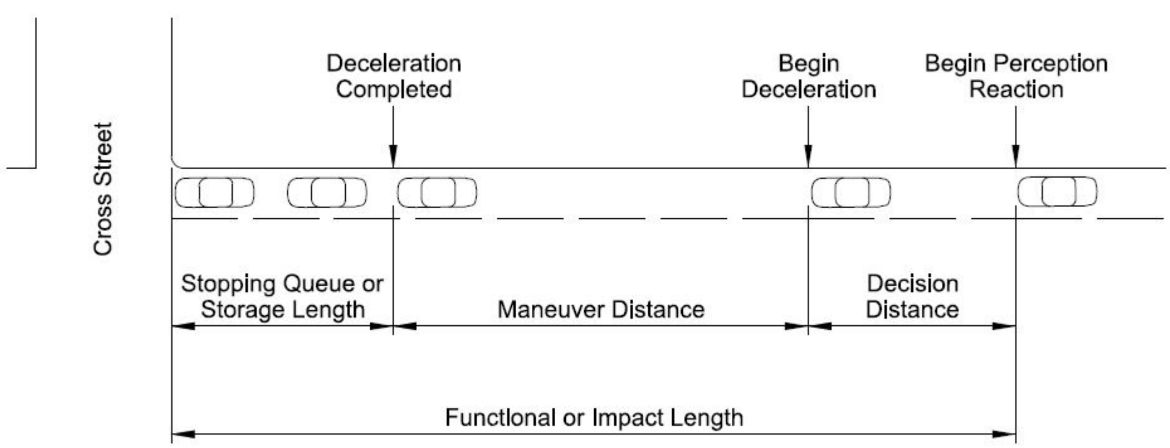


Figure 9-2 Elements of the Functional Area of an Intersection

9.2 TYPES AND EXAMPLES OF INTERSECTIONS

9.2.1 General Considerations

The basic types of at-grade intersections are the T-intersection (with multiple variations of angular approach), the four-leg intersection, the multi-leg intersections and roundabouts (see Chapter 19 of this Guide for roundabout design guidelines). In each particular case, the type is determined primarily by the number of intersecting legs, the topography, the traffic pattern, and the desired type of operation.

A basic intersection type can vary greatly in scope, shape, and degree of channelization. Once the type of intersection is established, the design controls and criteria covered in Chapter 2 and the elements of intersection design given in Chapter 3 as well as in this chapter must be applied to arrive at a suitable geometric plan.

Each type of intersection is discussed separately in Chapter 9 of the *PGDHS* **(1)**, and likely variations of each are demonstrated. It is not practical to discuss all possible variations, but the types demonstrated are sufficient to cover the general application of at-grade intersection design. Many other variations of types and treatment may be found in the NCHRP Report 279 **(2)**, which shows examples in detail that are not included in this Guide.

9.3 CAPACITY ANALYSIS

Capacity and level of service analysis is one of the most important considerations in the design of intersections. While highway level of service is typically defined by density, delay typically defines intersection level of service. Table 9-1 in Chapter 9 of the *PGDHS* **(1)** describes intersection level of service.

Optimum capacities can be obtained when at-grade intersections include auxiliary lanes, proper use of channelization, and traffic control devices. For more complete coverage of capacity of intersections, including procedures for making capacity computations, refer to Chapter 16 of the *Highway Capacity Manual* **(3)**.

9.4 ALIGNMENT AND PROFILE

9.4.1 General Considerations

Horizontal and vertical alignment and cross-sectional features affect driver and/or vehicle behavior at and on the approach to the intersection, and therefore are important design considerations. The horizontal and vertical alignment of the intersecting roads should permit users to readily discern and perform the maneuvers necessary to pass through the intersections safely and with a minimum of interference by other users.

As a rule, alignment and grade are subject to greater restriction at or near intersecting roads than on the open road. Their combination at or near the intersection must produce traffic lanes that are clearly visible to the operators at all times and plainly understandable for any desired direction of travel, free from unexpected hazards, and consistent with the portions of the highway just traveled.

9.4.2 Alignment

Both individual vehicle operations and the nature of vehicle conflicts are affected by the angle of intersection. Roads intersecting at acute angles require extensive turning roadway areas and tend to restrict visibility, particularly for drivers of trucks. When a truck turns on an obtuse angle, the driver has blind areas on the right of the vehicle. Acute-angle intersections increase the exposure time of the vehicles crossing the main traffic flow and may increase the accident potential.

Angles of 75 to 90 degrees are generally considered desirable. Although not desirable, a 60- degree angle is considered acceptable. New intersections should not include skewed angles less than 60 degrees without special design and control features to mitigate the effects of the skew. See Figure 9-22 in the *PGDHS* **(1)**. These may include more positive traffic control (all stop, traffic signals) and/or geometric improvements such as greater corner sight distance.

Geometric counter measures, although expensive, are generally the best solution in designing skewed-angle intersections. Reconstruction should reflect traffic patterns at the intersection as well as constraints such as available right of way. The practice of realigning roads intersecting at acute angles in the manner shown in Chapter 9 of the *PGDHS* **(1)** has proven to be beneficial.

Special care should be taken in designing intersections near horizontal curves. The driving task of tracking the curve takes up much of the driver's focus, leaving less attention for conflict resolution.

9.4.3 Profile

In general, grades for intersecting roads should be as flat as possible to provide for storage platforms and sight distance. Approach grades greater than 6 percent on low-speed highways should be avoided. For high-speed highways, grades greater than 3 percent should be avoided. Most vehicle operators are unable to judge the increase or decrease in stopping or accelerating due to steep grades. Their normal deductions and reactions thus may be in error at a critical time. Therefore, on grades steeper than 3 percent, the grade adjustment factors need to be applied to other design elements to produce conditions equivalent to those on level highways.

The effect of grade on acceleration can be expressed as a factor as shown in Table 10-5.

The profile gradelines and cross sections on the intersection legs should be adjusted for a distance back from the intersection, generally 20 feet, to provide a smooth junction and proper drainage. Normally, the gradeline of the major highway should be carried through the intersection, and that of the crossroad should be adjusted to it.

Changes from one cross slope to another should be gradual. Intersections with a minor road crossing a multilane divided highway with a narrow median and superelevated curve should be avoided whenever possible because of the difficulty in adjusting grades to provide a suitable crossing.

9.5 INTERSECTION CURVES

9.5.1 Widths for Turning Roadways at Intersections

The pavement and roadway 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 the intersection. Widths determined for turning roadways are also applied on all roadways within an intersection. The turning radii and the pavement cross slopes are functions of design speed and type of vehicles.

Pavement widths for turning roadways are classified as shown in Table 9-1 below.

Radius on Inner Edge of Pavement (ft)	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		
	Pavement Width (Feet) for Design Traffic Conditions								
	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
Width modification regarding edge treatment									
No stabilized shoulder	None			None			None		
Sloping Curb	None			None			None		
Vertical curb	Add 1 foot			None			Add 1 foot		
1 Side	Add 1 foot			None			Add 1 foot		
2 Sides	Add 2 feet			Add 1 foot			Add 2 feet		
Stabilized shoulder, one or both sides	Lane width for conditions B and C may be reduced to 12 feet where shoulder is 4 feet or wider.			Deduct shoulder width; minimum pavement width as under Case I.			Deduct 2 feet where shoulder is 4 feet or wider.		
A = Predominantly P vehicles, but some consideration for SU trucks B = Sufficient SU vehicles to govern design, but some consideration for semi trailer combination trucks C = Sufficient bus and combination-trucks to govern design									

Table 9-1[Table 3-29 of the *PGDHS* (1)] Design Widths of Pavements for Turning Roadways

9.5.1.1 Widths Outside Traveled Way

The roadway width for a turning roadway, as distinct from pavement width, includes the shoulders or equivalent lateral clearance outside the edges of pavement.

Table 9-2 (below) is a summary of the range of design values for the described general turning roadway conditions. On roadways without curbs or those with sloping curbs, the adjacent shoulder should be of the same type and section as that on the approach highway. The widths shown are for usable shoulders. Where roadside barriers are provided, the width indicated should be measured to the face of the barrier, and the graded width should be about 2 feet greater.

Turning movements should be checked with modeling software.

Turning Roadway Condition	**Shoulder Width or Lateral Clearance Outside of Traveled Way Edge (feet)**	
	Left	Right
Short length, usually within channelized intersection	2 to 4	2 to 4
Intermediate to long length, or in cut or on fill	4 to 10	6 to 12
Note: All dimensions should be increased where necessary for sight distance.		

Table 9-2[Table 3-30 of the *PGDHS* (1)] Range of Usable Shoulder Widths or Equivalent Lateral Clearances Outside Turning Roadways, Not on Structure

9.5.2 Minimum Designs for Sharpest Turns

After the lane width has been determined, site conditions along with traffic and island requirements will govern the curve selection. Generally, a three-centered curve is used to minimize the paved area and right of way requirements. The curve should be suitable for the anticipated truck traffic with the design for the commercial vehicle (SU) considered the desirable minimum.

Curbs along the edges of pavements of sharp intersection curves result in some restriction of vehicles making the turn. A design vehicle making its minimum turn will need to be maneuvered carefully to avoid scraping or jumping the curb. For this reason, when curbs are used, it is desirable to use somewhat flatter curves than those in minimum edge-of-pavement designs.

In the design of the edge of the pavement for the minimum path of a given design vehicle [see Figure 9-23 through 9-30 of the *PGDHS* **(1)**], it is assumed that the vehicle is properly positioned within the traffic lane at the beginning and end of the turn, i.e., 2 feet from the edge of pavement on the tangents approaching and leaving the intersection curve. Curve designs for edge of pavement conforming to this assumption, for passenger vehicles, single-unit trucks and buses, and semitrailer combinations are shown in Chapter 9 of the *PGDHS* **(1)**. The paths indicated, which are slightly greater than the minimum paths of nearly all vehicles in each class, are the minimums attainable at speeds less than 10 mph. In each case, these widths must be increased to address turning movements of vehicles over 10 mph operating speed. The wheel path should be 2 feet or more away from the edge of pavement throughout most of the turn, and at no point less than 9 inches. Although not shown separately in the figures in the *PGDHS* **(1)**, these edge designs also apply for left turn layouts such as a left turn to leave a divided highway at a very low speed. The designer should allow for an occasional large truck to turn by swinging wide and encroaching on other traffic lanes without disrupting traffic significantly. The designer should analyze the likely paths and encroachments that will result when a turn is made by a large vehicle.

The design should be modified where alignment conditions such as curvature prior to or at the end of the turn provide the assumed positioning. Superimposing the appropriate design-vehicle turning template is the most expeditious way to customize a design for special conditions.

Figures and data for three-centered curves (symmetrical and asymmetrical) are shown in Figures 9-3A, 9-3B, and 9-4, below, and Table 9-16 in the *PGDHS* **(1)**.

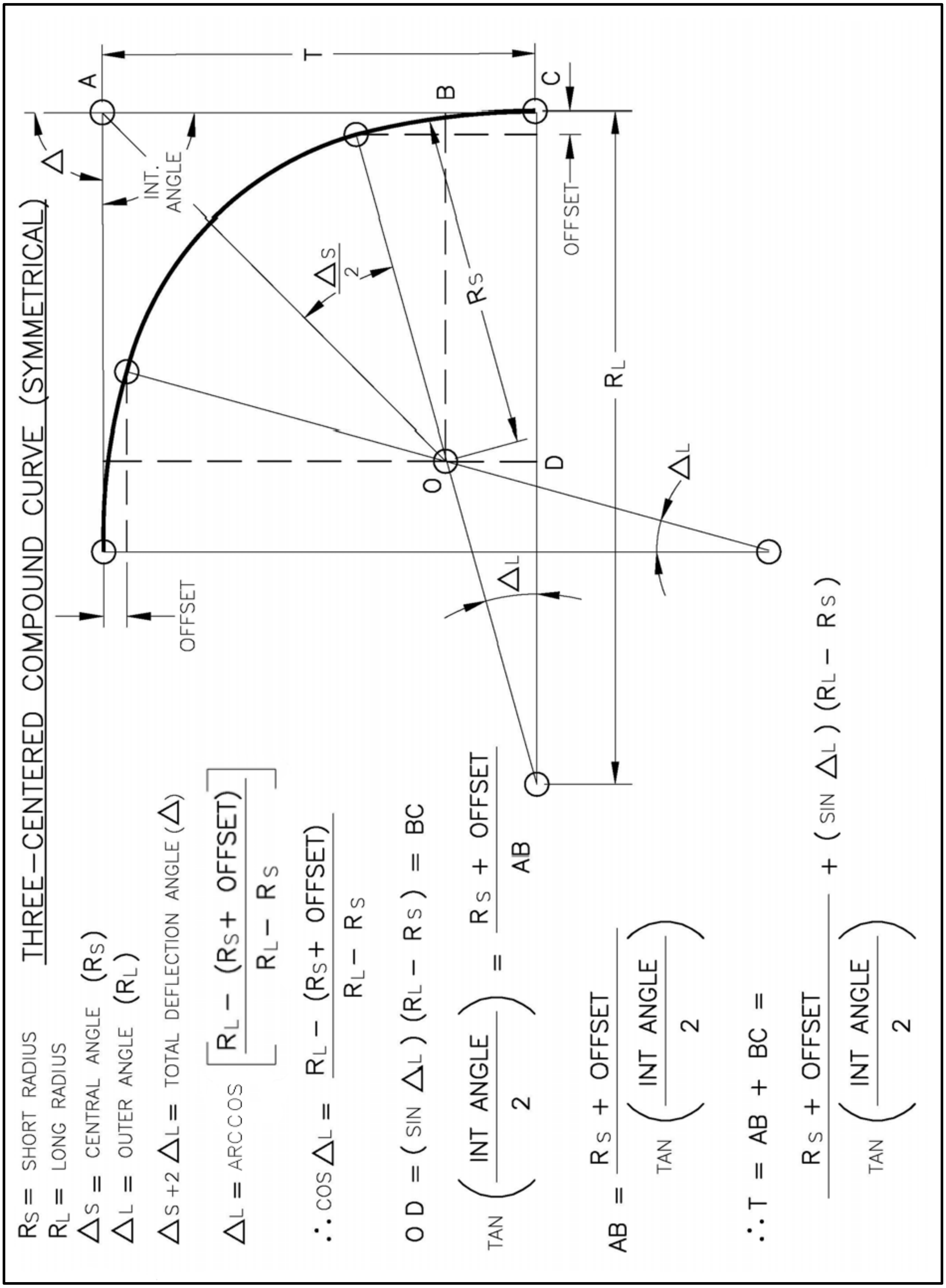


Figure 9-3A Three-Centered Compound Curve (Symmetrical)

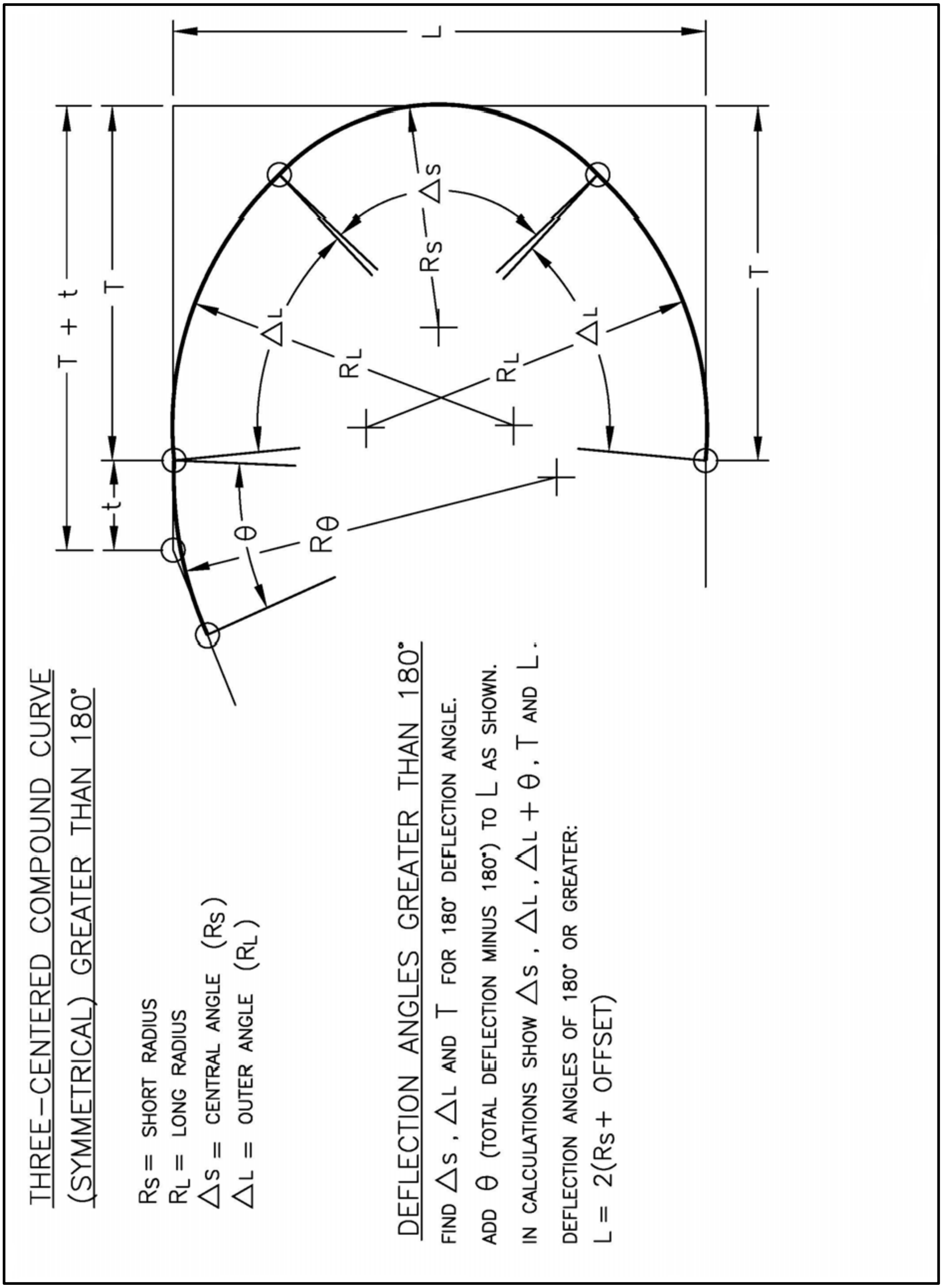


Figure 9-3B Three-Centered Compound Curve (Symmetrical) Greater than 180°

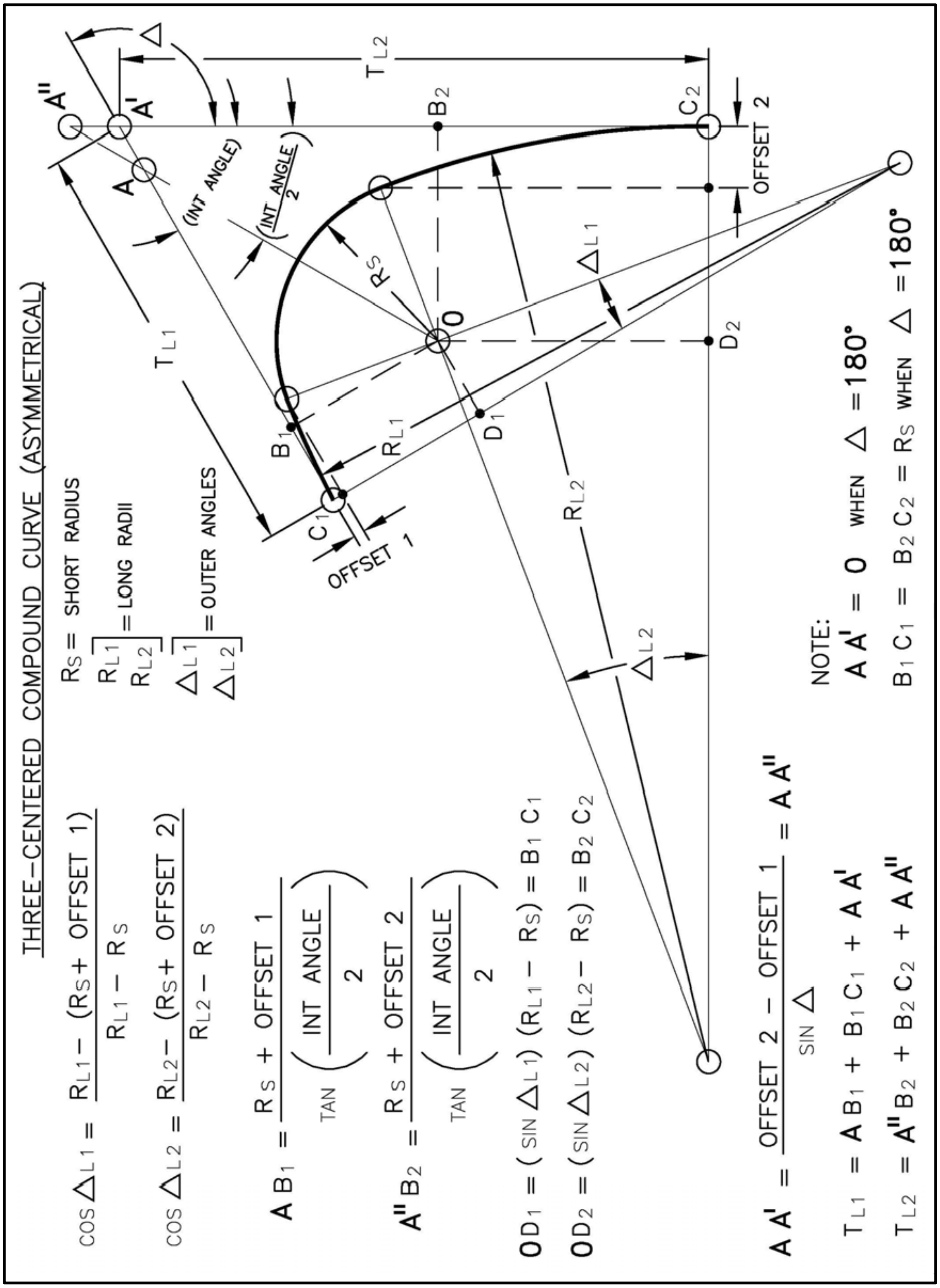


Figure 9-4 Three-Centered Compound Curve (Asymmetrical)

9.5.2.1 Design Vehicles

A description of the design vehicles used for intersection design is given in Chapter 2. For a thorough discussion and dimensions of the design vehicles, see Chapter 2 of the *PGDHS* **(1)**.

Select a design vehicle that is the largest vehicle that normally uses the intersection. The primary use of the design vehicle is to determine turning radius requirements for each leg of the intersection. It is possible for each leg to have a different design vehicle. Table 9-3 shows the minimum design vehicles. As justification to use a smaller vehicle, include a traffic analysis showing that the proposed vehicle is appropriate.

Intersection Type	Design Vehicle
Junction of Major Truck Routes	WB-67
Junction of State Highways	WB-67
Ramp Terminals	WB-67
Other Rural	WB-67
Industrial	WB-40
Commercial	SU(1)(2)
Residential	SU(1)(2)

(1) To accommodate pedestrians, the P vehicle may be used as the design vehicle if justification with a traffic analysis is documented.
(2) When the intersection is on a transit or school bus route, use the BUS design vehicle as a minimum.

Table 9-3 Intersection Design Vehicle

To minimize the disruption to other traffic, design the intersection to allow the design vehicles to make each turning movement without encroaching on curbs, opposing lanes, or same-direction lanes at the entrance leg.

Design each turning movement so the largest vehicle that is anticipated to regularly use the intersection can make the turn without leaving the paved shoulders or encroaching on a sidewalk. Use the WB-67 as the largest vehicle at all State highway to State highway junctions.

9.5.2.2 Effect of Curb Radii on Turning Paths

The widths for various angles of intersecting streets occupied by turning vehicles are shown in Table 9-17 and Figure 9-33 of the *PGDHS* **(1)**. When the angles increase, the streets must be very wide or a very large curb radius must be used. For this reason, three-centered curves are preferred for this type of situation.

Corner radii at intersections on arterial streets should satisfy the requirements of the drivers using them to the extent practical. The design should consider the amount of right of way available, the angle of intersection, number of pedestrians, width and number of lanes on the intersecting streets, and amount of speed reductions.

Consider the following:

- Radii of 15 to 25 feet are adequate for passenger vehicles. These radii may be provided at minor cross streets where there is little occasion for trucks to turn or at minor intersections where there are parking lanes. Where the street has sufficient capacity to retain the curb lane as a parking lane for the foreseeable future, parking should be restricted for appropriate distances from the crossing.
- Radii of 25 feet or more at minor cross streets should be provided on new construction and on reconstruction where space permits.
- Radii of 30 feet or more at major cross streets should be provided where feasible so that an occasional truck can turn without too much encroachment.
- Radii of 40 feet or more, and preferably three-centered compound curves or simple curves with tapers to fit the paths of appropriate design vehicles, should be provided where large truck combinations and buses turn frequently. Longer radii are also desirable where speed reductions would cause problems.
- Curb radii should be coordinated with crosswalk distances or special designs to make crosswalks safe for all pedestrians.

9.6 TURNING ROADWAYS & CHANNELIZATION

9.6.1 Oblique-Angle Turns

The designs given in Table 9-17 of the *PGDHS* **(1)** relating to minimum edge-of-pavement designs for turns at intersections are those suggested to fit the sharpest turns of the different design vehicles at oblique-angle intersections. For angles of turn less than 90 degrees, trucks also can turn on an inner edge of pavement designed for passenger vehicles with even less encroachment than that for the 90-degree turns. For turning angles of more than 90 degrees, the minimum design must be adjusted to ensure that all turning trucks remain within two lanes of pavement on each highway.

9.6.2 Development of Superelevation at Turning Roadway Terminals

Superelevation commensurate with curvature and speed seldom is practical at terminals where:

- A flat intersection curve results in little more than a widening of the through traffic pavement.
- It is desirable to retain the cross slope of the through pavement.
- There is a practical limit to the difference between the cross slope on the through pavement and that on the intersection curve.

9.6.3 General Procedure

The method of developing superelevation at turning roadway terminals is illustrated in Figure 9-44 through 9-47 of the *PGDHS* **(1)**. Superelevation over widening auxiliary lanes and over the turning roadway terminals should be gradual, cross slope controls are provided in Table 9-20 of the *PGDHS* **(1),** Maximum Algebraic Difference in Cross Slope at Turning Roadway Terminals.

9.7 ISLANDS

9.7.1 General Characteristics

An at-grade intersection in which traffic is directed into definite paths by islands is termed a channelized intersection.

An island is a defined area between traffic lanes for control of vehicle movements. Within an intersection, a median or an outer separation is considered an island. This definition makes evident that an island is no single physical type. It may range from an area delineated by a curb to a pavement area marked by paint.

Islands generally are included in intersection design (channelization) for one or more of the following purposes:

- Separation of conflicts
- Control of angle of conflict
- Reduction in excessive pavement areas
- Regulation of traffic and indication of proper use of intersection
- Arrangements to favor a predominant turning movement
- Protection of pedestrians (ADA requirements should be considered.)
- Protection and storage of turning and crossing vehicles
- Location of traffic control devices
- Access control

Islands generally are either elongated or triangular in shape and are situated in areas normally unused as vehicle paths. Their sizes and shapes vary materially from one intersection to another. Further variations occur at multiple and acute angle intersections. The dimensions depend on the particular intersection design. Islands should be located and designed to offer little hazard to vehicles, be relatively inexpensive to build and maintain, and occupy a minimum of roadway space, yet be commanding enough that motorists will not drive over them.

Where various intersections are involved in a given project and the warrants are sufficiently similar, in order to enhance driver expectancy, it is desirable to provide a common geometric design for each intersection.

Painted, flush medians and islands may be preferred to the curbed type under certain conditions, including the following:

- In lightly developed areas
- At intersections where approach speeds are relatively high
- Where there is little pedestrian traffic
- Where fixed-source lighting is not provided
- Where signals, signs, or lighting standards are not needed on the median or island

Painted islands may also be placed at the pavement edge. At some intersections, both curbed and painted islands may be desirable. All pavement markings should be reflectorized. The use of thermoplastic striping and other forms of long-life markings also may be desirable. For the various types and shapes of islands, see Figure 9-36 of the *PGDHS* **(1)**.

9.7.2 Island Size and Designation

Islands should be sufficiently large to command attention. The smallest curbed island that normally should be considered is one that has an area of approximately 50-square feet for urban and 75-square feet for rural intersections. However, 100-square feet are 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. Elongated or 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 to providing high visibility for the islands.

Curbed divisional islands introduced at isolated intersections on high-speed highways should be at least 100 feet and preferably several hundred feet in length. Details of triangular curbed islands and their size designation are shown in Chapter 9 of the *PGDHS* **(1)**.

9.7.3 Delineation

Islands should be delineated or outlined by a variety of treatments, depending on their size, location, and function. The type of area in which the intersection is located, rural versus urban, also governs the design. In a physical sense, islands can be divided into three groups:

- Raised islands outlined by curbs.
- Islands delineated by pavement markings, buttons, or raised (jiggle) bars placed on all paved areas.
- Non-paved areas 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-curbed islands is primarily addressed by curbs. Large curbed islands may be sufficiently delineated by color and texture contrast of vegetative cover, mounded earth, shrubs, guard posts, signs, or any combination of these. In rural areas, island curbs should nearly always be a sloping type, except where there is a definite need for a vertical curb, as at structures or pedestrian crossings. In special cases, vertical curbs are suitable, commonly of heights not more than 6 inches. Vertical or sloping curb could be appropriate in urban areas, depending on the condition. High-visibility curbs are advantageous at critical locations or on islands and roadway forks approached by high-speed traffic.

Curbed islands are sometimes difficult to see at night because of the glare from oncoming headlights or from distant luminaires or roadside businesses. Accordingly, where curbed islands are used, the intersection should have fixed-source lighting or appropriate delineation.

Delineation and warning devices are especially pertinent at approach ends of median curbed islands, which are usually in a direct line with approaching traffic. In rural areas, the approach should consist of a gradually widening center stripe. Although not as frequently obtainable, this approach should be strived for in urban areas also. Preferably, it should gradually change to a raised marking of color and texture contrasting with that of the traffic lanes or to jiggle bars that may be crossed readily even at considerable speed. This section should be as long as practicable.

9.7.4 Approach Treatment

The outline of a curbed island is determined by the edge of through traffic lanes and turning roadways, with lateral clearance to the curbed island sides. The points at the intersections of the curbed island are rounded or beveled for visibility and construction simplicity. 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, and traffic speed. Island curbs are introduced rather suddenly and should be offset from the edge of through traffic lanes even if they are sloping. A sloping curb at an island need not be offset from the edge of a turning roadway, except to reduce its vulnerability. Consider plowable end treatments. Vertical curbs should be offset from the edges of through and turning roadway pavements.

See Figure 9-40 in the *PGDHS* **(1)**.

9.7.5 Right-Angle Turns With Corner Islands

The turning roadway pavement should be wide enough to permit the outer and the inner wheel tracks of a selected vehicle to be within the edges of the pavement by about 2 feet on each side. Generally, the turning roadway pavement width should not be less than 14 feet. For designs of turning roadways of 90 degrees with minimum corner islands see Figure 9-38 of the *PGDHS* **(1)**.

9.7.6 Oblique-Angle Turns With Corner Islands

Minimum design dimensions for oblique-angle turns, determined on a basis similar to that for right-angle turns, are given in Table 9-4. Curve design for the inner edge of pavement, turning roadway pavement width, and the approximate island size are indicated for the three chosen design classifications described at the bottom of the table. For a particular intersection, the designer may choose from the designs shown in accordance with the size of vehicles, the volume of traffic anticipated, and the physical controls at the site.

		Three-Centered Curve			
Angle of Turn (degrees)	**Design Classification**	**Radii (ft)**	**Offset (ft)**	**Width of Lane (ft)**	**Approximate Island Size (sq ft)**
75	A	150-75-150	3.5	14	60
	B	150-75-150	5.0	18	50
	C	220-135-220	5.0	22	360
90	A	150-50-150	3.0	14	50
	B	150-50-150	11.0	21	150
	C	200-70-200	11.0	25	270
105	A	120-40-120	2.0	15	70
	B	150-35-150	11.5	29	65
	C	180-60-180	9.5	32	260
120	A	100-30-100	2.5	16	120
	B	150-30-150	10.5	33	130
	C	140-55-140	7.0	45	215
135	A	100-30-100	2.5	16	460
	B	150-30-150	10.0	38	395
	C	140-45-140	7.0	52	485
150	A	100-30-100	2.5	16	1400
	B	150-30-150	9.0	42	1350
	C	160-40-160	6.0	53	1590

A - Primarily passenger vehicles; permits occasional design single-unit truck to turn with restricted clearances.
B - Provides adequately for SU-9 [SU-30] and SU-12 [SU-40] design vehicles; permits occasional WB-19 [WB-62] design vehicles to turn with slight encroachment on adjacent traffic lanes.
C - Provides fully for WB-19 [WB-62] design vehicle

Asymmetric three-centered compound curves and straight tapers with a simple curve can also be used without significantly altering the width of roadway or corner island size.

Table 9-4 [Table 9-18 of the *PGDHS* (1)] Typical Designs for Turning Roadways

In Table 9-4, design values are not given for angles of turn less than 75 degrees. Turning roadways for flat-angle turns involve relatively large radii and are not considered in the minimum class. Such arrangements require individual design to fit site controls and traffic conditions. For angles of turn between 75 and 120 degrees, the designs are governed by a minimum island, providing for turns on more than minimum turning radii. For angles of 120 degrees or more, the sharpest turning paths of the selected vehicles, and arrangements of curves on the inner edge of traveled way to fit these paths, generally control the design. The resulting island size is greater than the minimum. The size of the islands for the large turning angles in Table 9-4 is indicative of the otherwise unused and uncontrolled areas of traveled way that are eliminated by the use of islands.

9.8 INTERSECTION SIGHT DISTANCE

9.8.1 General Considerations

Conflicts between vehicles at intersections are resolved by providing appropriate sight distances and traffic controls. The avoidance of accidents and the efficiency of the traffic operations still depend on the judgment, capabilities, and response of the individual driver. The sight distance considered safe is directly related to vehicle speeds and to the resultant distances traversed during perception, reaction time, and braking.

Each vehicle approaching an intersection must have an unobstructed sight distance along all legs of the cross road and across its included corners for a distance sufficient to allow the drivers to see each other in time to prevent a collision.

Each driver can either:

- Accelerate
- Slow down
- Stop

After a vehicle has stopped at an intersection, the driver must be able to make a safe departure through the intersection area. Figures 9-5A and 9-5B indicate these maneuvers as well as the sight distances that must be provided for vehicles approaching on the major highway from either direction. Distance “b” is the length of roadway traveled by the respective vehicle on the major roadway during the time required for the stopped vehicle to depart from its stopped position and either cross the intersection or to turn onto the major roadway.

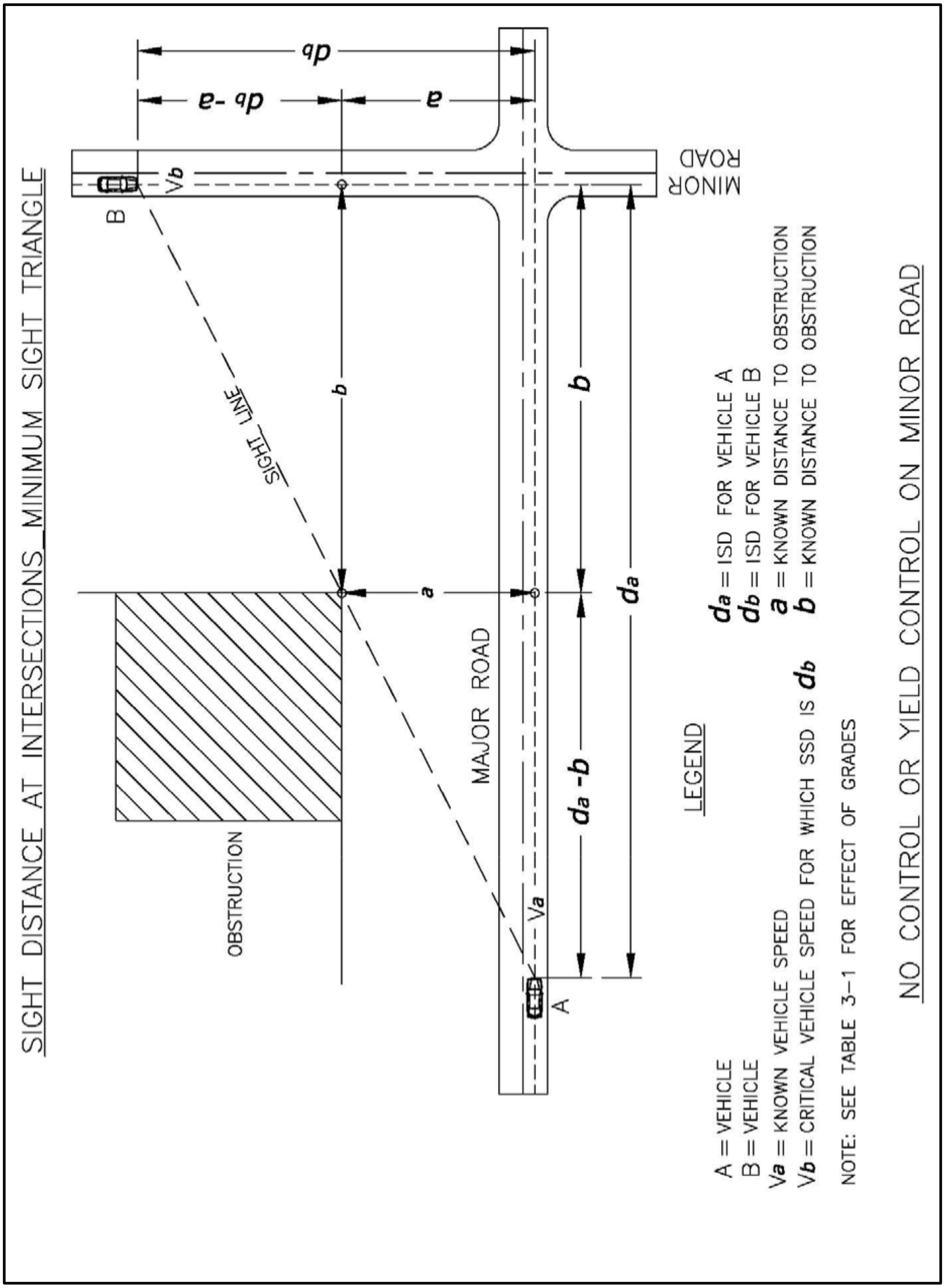


Figure 9-5A Sight Distance at Intersections, Minimum Sight Triangle

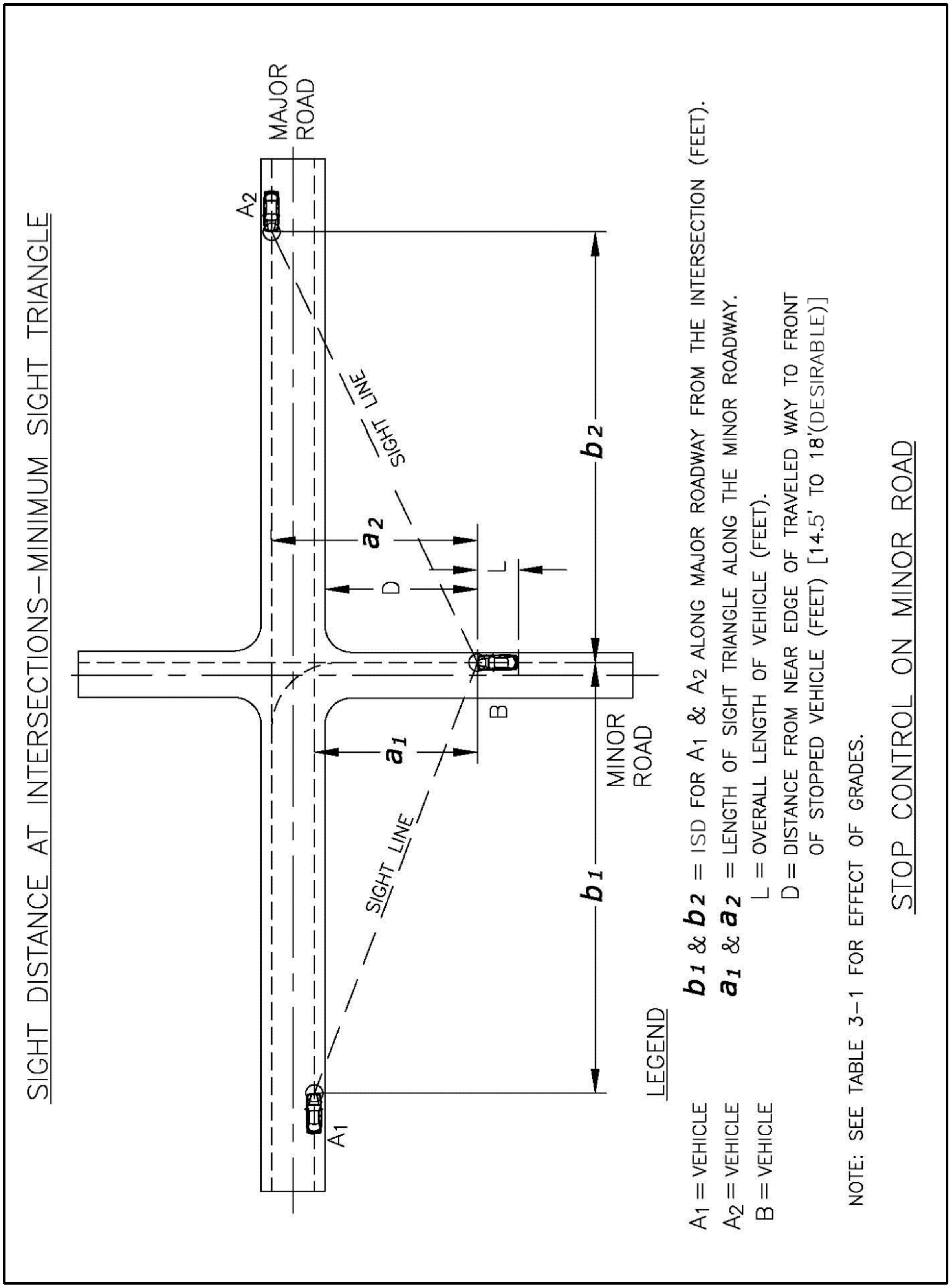


Figure 9-5B Sight Distance at Intersections, Minimum Sight Triangle

9.8.2 Intersection Control

The recommended dimensions of the sight triangles vary with the type of traffic control used at an intersection because different types of control impose different legal constraints on drivers and, therefore, result in different driver behavior. Procedures to determine sight distances at intersections are presented in Chapter 9 of the *PGDHS* **(1)** for each of the Cases below:

Case A – Intersections with no control (not used on State highways)

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

****Note: Case B2 applies to signalized intersections including ramp terminals where right turn on red is permitted.***

Case C – Intersections with yield control on the minor road

Case C1 – Crossing maneuver from the minor road
Case C2 – Left and right turn maneuvers

Case D – Intersections with traffic signal control

Case E – Intersections with all-way stop control

Case F – Left turns from the major road

9.8.3 Effect of Skew

See section 9.5.4 of the *PGDHS* **(1)**.

9.9 STOPPING SIGHT DISTANCE AT INTERSECTIONS FOR TURNING ROADWAYS

The values for stopping sight distance as computed in Chapter 3 for open highway conditions are applicable to turning roadway intersections of the same design speed.

See also Table 9-21 in the *PGDHS* **(1)** and Table 3-1.

9.10 DESIGN TO DISCOURAGE WRONG-WAY ENTRY

An inherent problem of intersections is the possibility of a driver entering one of the exit terminals from the crossroad and proceeding along the major highway in the wrong direction in spite of signing. This wrong-way entrance is becoming more of a problem with the increased number of intersections. Attention to several details of design at the intersection can discourage this hazardous maneuver.

Details of designs to discourage wrong-way entry are shown in Figures 10-56 and 10-57 of the *PGDHS* **(1)**.

9.11 SUPERELEVATION FOR CURVES AT INTERSECTIONS

9.11.1 General Design Considerations

Providing proper drainage and the matching of grades and cross slopes can create problems on low-speed turning roadways. See Tables 9-19 and 9-20, and Figures 9-44 to 9-47 of the *PGDHS* **(1)**.

9.12 CHANNELIZATION

Channelization is the separation or regulation of conflicting traffic movements into delineated paths of travel by traffic islands or pavement marking to facilitate the safe and orderly movements of vehicles, bicycles, and pedestrians.

9.13 MEDIAN OPENINGS

See Tables 9-25 through 9-27 and Figures 9-55 through 9-58 in the *PGDHS* **(1)**.

Medians were discussed in Chapter 4 chiefly as an element of the cross section. General ranges in width were given, and the width of the median at intersections was treated briefly. For intersection conditions, the median width, the length of the opening, and the design of a median opening and median ends should be based on traffic volumes and type of turning vehicles. Cross and turning traffic must operate in conjunction with the through traffic on the divided highway. This requirement makes it necessary to know the volume and composition of all movements occurring simultaneously during the design peak hours.

The design of a median opening becomes a matter of considering what traffic is to be accommodated, choosing the design vehicle to use for layout controls for each cross and turning movement, investigating whether larger vehicles can turn without undue encroachment on adjacent lanes, and finally checking the intersection for capacity. If the capacity is exceeded by the traffic load, the design must be expanded, possibly by widening or otherwise adjusting widths for certain movements.

9.13.1 Control Radii for Minimum Turning Paths

An important factor in designing median openings is the path of each design vehicle making a minimum left turn at 10 to 15 mph. Where the volumes and types of vehicles making the left turn movements call for higher than minimum speed, the design may be made by using a radius of turn corresponding to the speed deemed appropriate. However, the minimum turning path at low speed is needed for minimum design and for testing layouts developed for one vehicle with an occasional larger vehicle.

The paths of design vehicles making right turns are shown in Chapter 2 of the *PGDHS* **(1)**. Any differences between the minimum turning radii for left turns and those for right turns are small and are insignificant in highway design. Minimum 90-degree left-turn paths for design vehicles are shown in Figure 9-54 of the *PGDHS* **(1)**. Turning templates, whether electronic or transparent, for various design vehicles should be utilized.

By considering the range of radii for minimum right turns and the need for accommodation of more than one type of vehicle at the usual intersections, the following control radii can be used for minimum practical design of median ends:

- A control radius of 40 feet accommodates P vehicles suitably and occasional SU-30 vehicles with some swinging wide.
- One of 50 feet accommodates SU-30 vehicles and occasional WB-40 vehicles with some swinging wide.
- One of 75 feet accommodates WB-40
- One of 130 feet accommodates WB-62 and occasionally WB-67

See Figures 9-55 to 9-58 of the *PGDHS* **(1)**.

9.13.2 Shape of Median End

One form of a median end at an opening is a semicircle. This simple design is satisfactory for narrow medians. For medians greater than about 10 feet in width the bullet nose is superior to the semicircular end. Consider plowable end treatments.

Alternate minimum designs for median ends to fit the design control radii of 40, 50, 75 and 130 feet are shown in Chapter 9 of the *PGDHS* **(1)**. See Figures 9-55 to 9-58 of the *PGDHS* **(1)**.

9.13.3 Median Openings Based on Control Radii for Design Vehicles

A 130-foot control radius is sufficiently large to accommodate a WB-67 design vehicle.

9.13.4 Effect of Skew

A control radius for design vehicles as the basis for minimum design of median openings results in lengths of openings that increase with the skew angle of the intersection. See Tables 9-28 and 9-29 of the *PGDHS* **(1)** for details of the effect of skewed crossings on the length of median openings.

In general, the asymmetrical bullet nose end is preferable.

9.14 ABOVE MINIMUM DESIGNS FOR DIRECT LEFT TURNS

See Figure 9-59 of the *PGDHS* **(1)**.

9.15 INDIRECT LEFT TURNS AND U-TURNS

9.15.1 General Design Considerations

At intersections where the median is too narrow to provide a lane for left-turning vehicles and the traffic volumes or speeds, or both, are relatively high, safe, efficient operation is particularly troublesome. Vehicles that slow down or stop in a lane primarily used by through traffic to turn left greatly increase the potential for rear-end collisions. The necessity to turn left in the urban or heavily developed residential or commercial sectors also presents serious problems with respect to safety and efficient operation. Chapter 9 of the *PGDHS* **(1)** shows several options that may be considered for indirect left turns on high-speed/high-volume highways.

In special circumstances, there may be a need to include U-turns in the design if alternative access to businesses or homes is not available.

Normally U-turns should not be permitted from the through lanes. For a satisfactory design for U-turn maneuvers, the width of the highway, including the median, should be sufficient to permit the design vehicle to turn from an auxiliary left-turn lane in the median into the lane next to the outside shoulder or outside curb and gutter on the roadway of the opposing traffic lanes.

9.16 FLUSH OR TRAVERSABLE MEDIANS

The foregoing discussion on design for indirect left turns U-turns with raised curb medians brings into focus the difficulties involved in providing access to abutting property, especially where such access is by commercial vehicles. These conditions are common in commercial and industrial areas where property values are high and right of way for wide medians is difficult to acquire.

One method for solving this left-turn conflict problem while maintaining access to roadside activities is the use of continuous two-way left-turn lanes. Two-way left-turn lanes should be considered when a history of midblock accidents involving left-turning vehicles exists, driveways are closely spaced, and/or strip development or multiple-unit residential land use exists along the corridor.

A number of studies have evaluated the cost-effectiveness of two-way left-turn lanes. Based on these studies, the following warrants and guidelines are suggested for their application:

- Annual Average Daily Traffic (AADT) - four lane – between 10,000 and 20,000 vehicles, or two lane – between 5,000 and 12,000 vehicles.
- Left-turn volumes - 70 midblock turns per 1,000 feet, or at least 20 percent of total volume in peak hour.
- Minimum length - 1,000 feet, or two to three blocks

In general, two-way left-turn lanes can be considered when operating speeds are 50 mph or less. Lane widths in use range from 10 to 16 feet. Table 9-5 provides suggested lane widths for various types of highways. New designs should strive for lane widths of 12 to 14 feet.

Lane Widths for Continuous Two-Way Left-Turn Lanes		
Prevailing Speed	Lane Use (Vehicle Type)	Appropriate Width of Lane (ft)
25 to 30	Residential, business (passenger cars)	10 absolute minimum, 12 desirable
30 to 40	Business (passenger cars, some trucks)	12 minimum, 14 desirable
	Industrial (many large trucks)	14 to 16
40 to 50	Business	12 minimum, 14 desirable

Table 9-5 [From NCHRP Report 279 (2)] Lane Widths for Continuous Two-Way Left-Turn Lanes

Conversion of existing cross sections is generally more constrained. However, narrow widths should be avoided except for low-speed streets. Lane widths should not be so great, however, that shared use of the lane (i.e., side-by-side) by opposing drivers is created.

The two-way left turn lane is generally continued through minor or unsignalized intersections. For signalized intersections or those controlled by four-way stops, it is generally advisable to restrict entry into the lane for a reasonable distance from the intersection using pavement markings.

9.17 AUXILIARY (SPEED CHANGE) LANES

The primary purpose of auxiliary lanes at intersections is to provide storage for turning vehicles, both left and right. A secondary purpose is to provide space for turning vehicles to decelerate from the normal speed of traffic to a stopped position in advance of the intersection or to a safe speed for the turn in case a stop is unnecessary. Additionally, auxiliary lanes may be provided for bus stops or for loading and unloading passengers from passenger cars.

A speed change lane is an auxiliary lane, including tapered areas, primarily for the acceleration or deceleration of vehicles entering or leaving the through traffic lanes. The terms "speed-change lane," "deceleration lane," or "acceleration lane," as used here, apply broadly to the added pavement joining the traveled way of the highway or street with that of the turning roadway and do not necessarily imply a definite lane of uniform width.

Speed-change lanes may be justified on high-speed and on high-volume highways where a change in speed is necessary for vehicles entering or leaving the through traffic lanes.

9.17.1 General Design Considerations

Desirably, the total length of the auxiliary lane should be the sum of the length for three components. Where intersections occur as frequently as four per mile, it is customary to forego most of the deceleration length and to provide only the storage length plus taper. Each component of the auxiliary length is discussed in the following section. Where geographically possible, a continuous auxiliary lane shall be established between accesses in instances where speed change lanes overlap, or are separated by less than 300 feet or half their length (whichever is shorter). Figure 9-6 illustrates basic auxiliary lane elements.

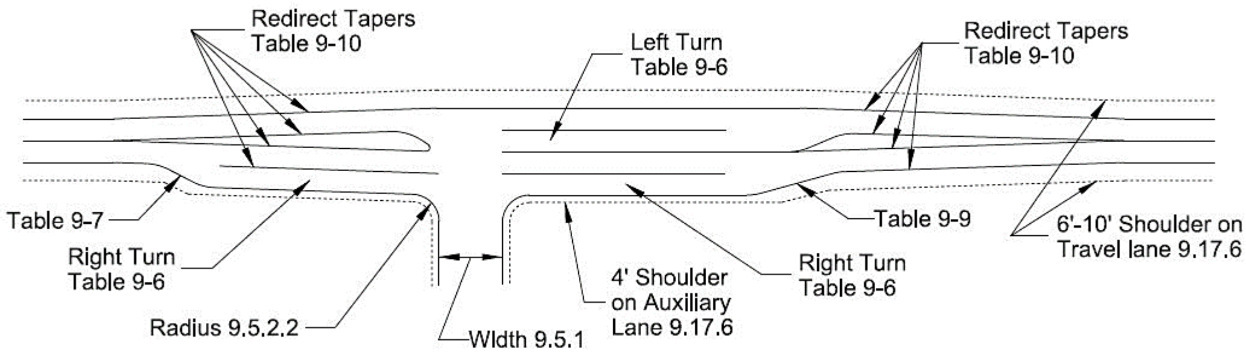


Figure 9-6 [Figure 4-1 of the *State Highway Access Code* (2)] Information Guide to Basic Auxiliary Lane Elements

Auxiliary lanes should be at least 10-feet wide and desirably should equal that of the through lane. Where curbing is to be used adjacent to the auxiliary lane, an appropriate curb offset should be provided. The length of the auxiliary lanes for turning vehicles consists of three components:

- Deceleration length
- Storage length
- Entering taper

Warrants for the use of auxiliary lanes cannot be stated definitely. Observations and considerable experience with speed change lanes have led to the following general conclusions:

- Speed-change lanes are warranted on high-speed and high-volume highways where a change in speed is necessary for vehicles entering or leaving the through traffic lanes.
- All drivers do not use speed change lanes in the same manner -- some use little of the available facility. As a whole, however, these lanes are used sufficiently to improve the overall safety and operation of the highway.
- Use of speed change lanes varies with volume; the majority of drivers use the lanes during high volumes.
- The directional type of speed-change lane consisting of a long taper fits the behavior of most drivers and does not require maneuvering on a reverse curve path.
- Deceleration lanes on the approaches to intersections that also function as storage lanes for turning traffic are particularly advantageous, and experience with them generally has been favorable. Such lanes reduce hazards and increase capacity.

Deceleration lanes always are advantageous, particularly on high-speed roads, because the driver of a vehicle leaving the highway has no choice but to slow down on the through traffic lane if a deceleration lane is not provided. The failure to brake by the following drivers because of a lack of alertness causes many rear-end collisions.

Acceleration lanes are not always necessary at stop-controlled intersections where entering drivers can wait for an opportunity to merge without disrupting traffic. Acceleration lanes are advantageous on highways without stop control and on all high-volume roads even with stop sign control where openings between vehicles in the peak hour traffic streams are infrequent and short.

The use and design of speed change lanes differ between functional roadway classifications. Table 9-6 defines the functional roadway classification speed change lane design components by access category. Consult with the Region Traffic Engineer who will determine the need for auxiliary lanes by calculating the highway capacity and determining if the capacity could be improved by the addition of acceleration and deceleration lanes.

Access Category	Left Turn Deceleration Lane	Right Turn Deceleration Lane	Acceleration Lane
F-W	Design Must Meet Federal Interstate Standards, and No Less Than E-X		
E-X	Taper + Decel. Length + Storage	Taper + Decel. Length	Accel. Length + Taper
R-A	*Decel. Length + Storage	*Decel. Length	*Accel. Length
R-B	*Decel. Length + Storage	*Decel. Length	*Accel. Length
NR-A	*Decel. Length + Storage	*Decel. Length	*Accel. Length
NR-B	Taper + Storage	Taper + Storage	*Accel. Length
NR-B > 40 MPH	*Decel. Length	*Decel. Length	*Accel. Length
NR-C	Taper + Storage	Taper + Storage	*Accel. Length
NR-C > 40 MPH	*Decel. Length	*Decel. Length	*Accel. Length
F-R	Taper + Storage	Taper + Storage	*Accel. Length
F-R > 40 MPH	*Decel. Length	*Decel. Length	*Accel. Length
* Taper Length is Included Within Stated Accel. or Decel. Length			

Table 9-6 [Figure 4-5 of the *State Highway Access Code* (2)] Components of Speed Change Lane Length

9.17.2 Deceleration Length

The functional area of an intersection with relation to the deceleration lane length is shown in figure 9-7. This graphic illustrates the upstream functional area of an intersection with includes the three components: perception-reaction distance, deceleration lane length, storage length. The physical length of the deceleration lane includes the taper length, the deceleration length and the storage length.

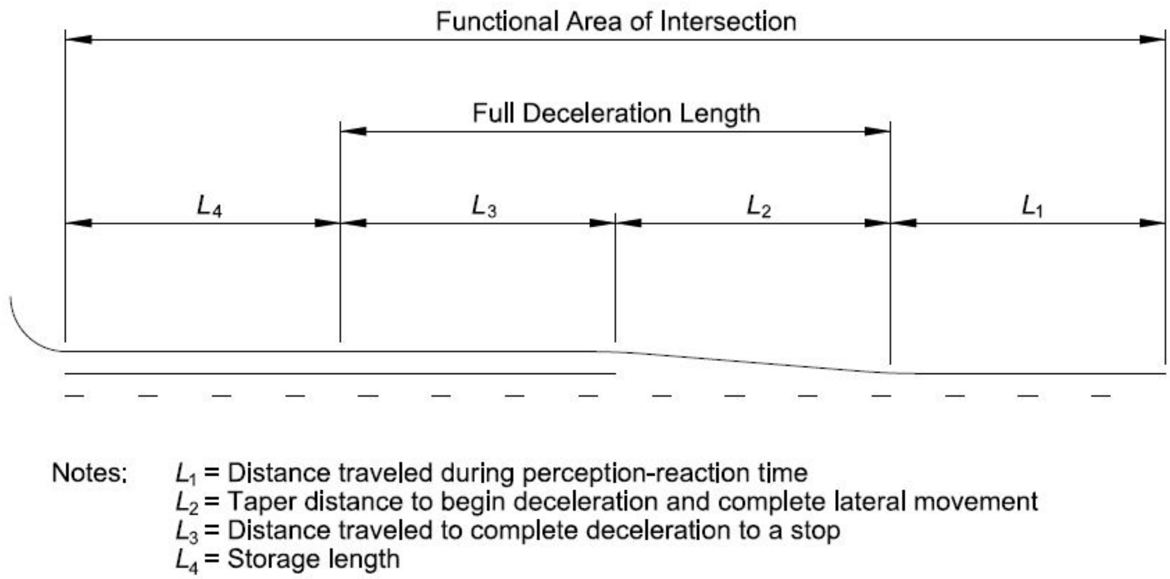


Figure 9-7 [Figure 9-48 of the *PGDHS* (1)] Functional Area Upstream of an Intersection Illustrating Components of Deceleration Lane Length

Table 9-7 represents the estimated distances to maneuver from the through lane into a turn bay and brake to a stop. It is not always practical to provide the full deceleration length of the auxiliary lane for deceleration due to constrained ROW or available distance between intersections. In these cases, part of the deceleration by drivers should occur before entering the auxiliary lane.

Speed (MPH)	20	30	40	50	60	70
Distance[a] (Feet)	70	160	275	425	605	820
[a] Rounded to the nearest 5 feet						

Table 9-7 [Table 9-22 of the *PGDHS* (1)] Desirable Full Deceleration Lengths

9.17.3 Storage Length

The auxiliary lane should be sufficiently long to store the number of vehicles likely to accumulate during a critical period. The storage length should be sufficient to avoid the possibility of left-turning vehicles stopping in the through lanes. The storage length should be sufficiently long so that the entrance to the auxiliary lane is not blocked by vehicles standing in the through lanes waiting for a signal change or for a gap in the opposing traffic flow.

At unsignalized intersections, the storage length, exclusive of taper, may be based on the number of turning vehicles likely to arrive in an average two-minute period within the peak hour. The two-minute waiting time may need to be changed to some other interval that depends largely on the opportunities for completing the left-turn maneuver. These intervals, in turn, depend on the volume of opposing traffic. Where the volume of turning traffic is high, traffic signal warrants should be checked.

There are several techniques used to determine the necessary storage length. The required storage length for an unsignalized intersection is in accordance with Table 9-8.

Turning Vehicles Per Peak Hour	Below 30	30	60	100	200	300
Required Storage Length (ft)	25	*40	*50	100	200	300
*Minimum storage length is 100 ft when trucks equal or exceed 10 percent of turning vehicles.						

Table 9-8 [*State Highway Access Code* Table 4-8 (4)] Storage Lengths for Auxiliary Lanes

A left-turning volume of 200 vehicles per hour, or more, could not complete the turn without difficulty unless the volume of opposing traffic during the same hour is about 88 or less. Turning volumes in this range usually require special design or traffic signal control. Storage lengths for signalized intersections may be determined from highway capacity nomographs in the Highway Capacity Manual **(3)**.

Check with local agencies to ascertain their established minimum lengths for lanes with low turn volumes.

The important factors which determine the length needed are:

- The design year volume for the peak hour.
- An estimate for the number of cycles per hour if the location is signalized.
- The type of signal phasing and timing that will control the left-turn movement. Coordinate with the Traffic Engineer.
- To reduce the total length of queues formed in the left-turn lane, it is a desirable practice to allow "permissive" turns following "protected" turn phases. Permissive turns are made when gaps in opposing traffic occur and can increase the capacity of a turn lane from 20 to 50 percent. Permissive turns are not allowed where multiple left-turn lanes exist.

9.17.4 Acceleration Length

Acceleration lanes are used when there is a free-flow right and the *Highway Capacity Manual* **(3)** dictates. For warrants see Chapter 11. For design, see Table 9-9.

Design Speed (MPH)	30	35	40	45	50	55	60	65	70	75
Acceleration Length (Feet)	180	280	360	560	720	960	1200	1410	1620	1790
*These approximate lengths are based on grades less than 3 percent.										

Table 9-9 [Table 10-3 of the *PGDHS* (1)] Desirable Acceleration Length From Stop Condition

Provision for acceleration clear of the through traffic lanes is a desirable objective on arterial roads and streets and should be incorporated into design whenever feasible and practicable (see the *State Highway Access Code* **(4)** for guidance on warrants). The total length required is that needed for a safe and comfortable speed required to enter the through lanes. Acceleration requirements are as show in Figure 10-69 and Tables 10-3 to 10-4 of the *PGDHS* **(1)**.

9.17.5 Speed Change Lane Width

Speed change lane widths shall be a minimum of 11 feet, not including the gutter pan or shoulder, whenever posted speeds are greater than 40 mph, or when truck volumes exceed nine percent. Ten foot lanes may be used in instances where the posted speed limit is less than 45 mph, and truck volumes are less than ten percent, so long as the local design standards allow. In instances where adjacent travel lanes are 12 feet wide, the speed change lane shall be designed at 12 feet wide.

9.17.6 Shoulder Width Along Speed Change Lanes Where Curbs are not Present

Shoulders must be present in all locations where curb and gutter does not exist. Shoulders adjacent to through travel lanes should be six feet wide, but no less than the existing shoulder width. Shoulders located on highways designated as part of the National Highway System, should be no less than ten feet wide. Shoulders along speed change lanes shall be a minimum of four feet wide.

9.17.7 Taper

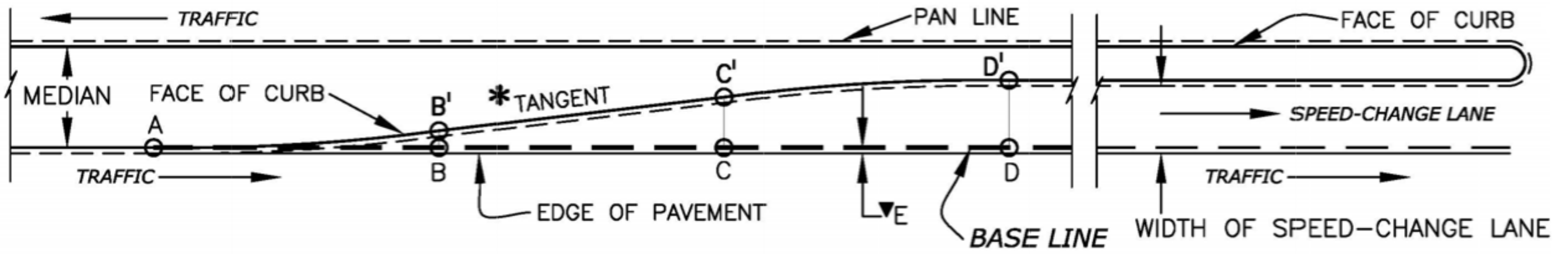
To develop the width needed for auxiliary lanes, a transition must be effected. This transition, or taper, allows a driver to recognize that an exclusive lane is being developed and also allows some deceleration to occur prior to entering the storage lane itself.

Design configurations for straight-line and curved tapers are shown in Chapter 9 of the *PGDHS* **(1)**. Recommended taper ratios for speed-change lanes are given in Table 9-10.

Posted Speed (MPH)	25	30	35	40	45	50	55	60 [a]	65 [a]	70 [a]
Taper Ratio [b]	7.5:1	8:1	10:1	12:1	13.5:1	15:1	18.5:1	25:1	25:1	25:1

[a]Uniform 50:1 to 70:1 tapers are recommended where lengths of acceleration lanes exceed 1300 feet.
[b]Taper Length equals taper ratio times lane width.

Table 9-10 [*State Highway Access Code* Table 4-6 (4)] Taper Length and Ratio for Parallel-Type Entrance



NOTES

1. AD = TAPER LENGTH
2. AB = BC = CD = 1/3 AD
3. AB' & C'D' ARE PARABOLIC CURVES.
4. THE INCLUDED TABLE GIVES OFFSETS FROM A ***BASE LINE*** PARALLEL TO THE EDGE OF PAVEMENT TO THE FACE OF CURB AT INTERVALS MEASURED FROM POINT A .

OFFSET DISTANCE (FEET)		DESIGN SPEED (MPH)			
		30	40	50	
		LENGTH OF TAPER (FT)			
		60	90	150	
DD' =10'	DD' =12'	DISTANCE FROM POINT A (FEET)			
0	0	---	---	---	
0.16	0.19	5	7.5	12.5	
0.62	0.75	10	15.0	25.0	
1.41	1.69	15	22.5	37.5	
2.50	3.00	20	30.0	50.0	DATA FOR POINT **B'**
5.00	6.00	30	45.0	75.0	
7.50	9.00	40	60.0	100.0	DATA FOR POINT **C'**
8.59	10.31	45	67.5	112.5	
9.38	11.25	50	75.0	125.0	
9.84	11.81	55	82.5	137.5	
10.00	12.00	60	90.0	150.0	

5. * WHERE EDGE OF PAVEMENT IS A CURVE, NEITHER ***BASE LINE*** NOR TAPER BETWEEN **B'** & **C'** WILL BE A TANGENT. USE PROPORTIONAL OFFSETS FROM **B'** TO **C'**
6. ▼ THE ADOPTED OFFSET E IS USUALLY ONE FOOT ALONG OUTSIDE EDGE OF PAVEMENT BUT MAY VARY IN SOME CASES.
7. THE ABOVE TABLE MAY ALSO BE USED WITH A PAINTED MEDIAN TO DELINEATE THE STORAGE AREA.

Figure 9-8 Speed-Change Lane Taper for Continuously Curbed Medians

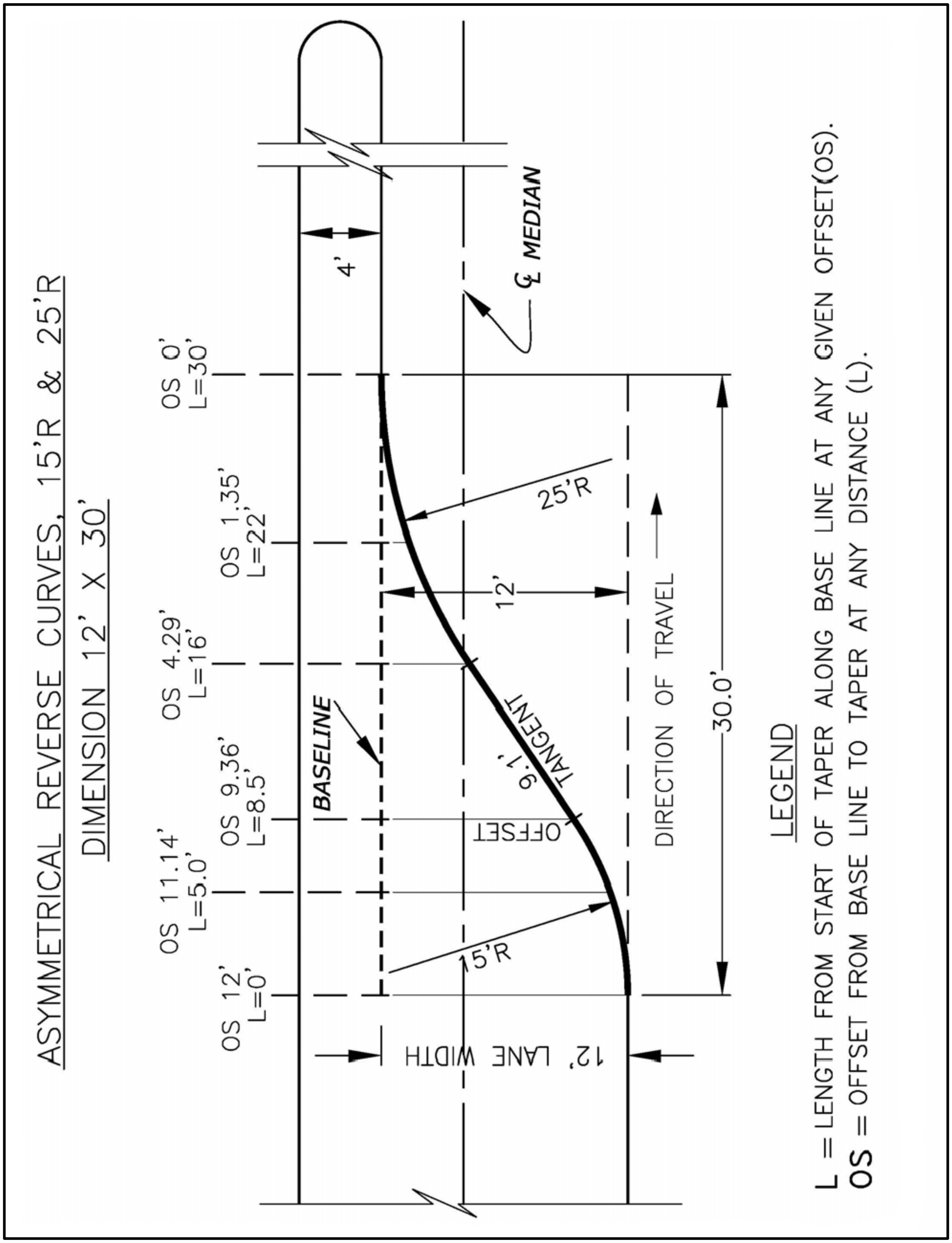


Figure 9-9 (CDOT) Median Bay Taper

Median Bay tapers (asymmetrical reverse curves) may be used for deceleration transition tapers. Use of a bay taper and auxiliary lane striping will reduce drifting of the through vehicles into the deceleration lane. Where horizontal or crest vertical curves exist, consider using a bay taper for more visible definition.

9.17.7.1 Elements of Left-Turn Design (Redirect Taper)

Section 3B.10 “Approach Markings for Obstructions,” in the *Manual on Uniform Traffic Control Devices (MUTCD)* **(5)** recommends the following for design speeds equal to or greater than 45 mph:

$$L = WS \tag{9-1}$$

Where,

L = length of taper, ft
S = design speed, mph
W = offset, ft

For design speeds less than 45 mph, the *MUTCD* recommends:

$$L = \frac{WS^2}{60} \tag{9-2}$$

The departure taper should be designed in concert with the left-turn lane on the opposite approach. The departure taper should begin opposite the beginning of the left-turn lane, and continue to a point at least opposite the approach taper. Extension of the departure taper beyond the approach nose of a raised median channelization is recommended wherever possible.

9.17.8 Median Left-Turn Lanes

Accommodation of left turns in many cases is the critical factor in design of intersections. Provisions for left-turn lanes greatly influence both level of service and intersection safety.

A median lane provides refuge for vehicles awaiting an opportunity to turn, and thereby keeps the highway traveled way clear for through traffic. The width, length, and general design of median lanes are similar to those of any other deceleration lane but their design includes some additional features. Examples of median left-turn channelization are shown in Figures 9-10A and 9-10B.

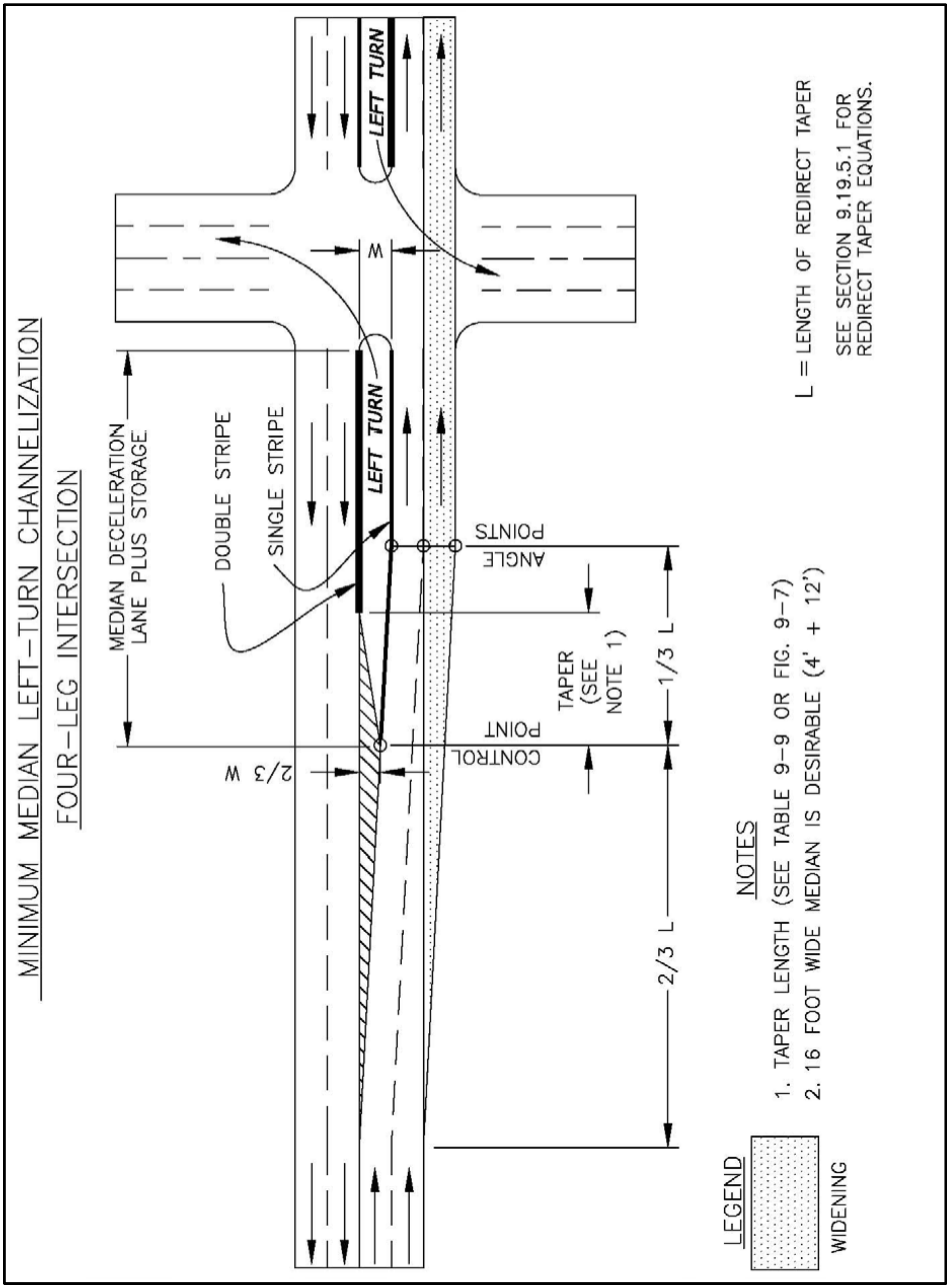


Figure 9-10A Minimum Median Left-Turn Channelization, Four–Leg Intersection

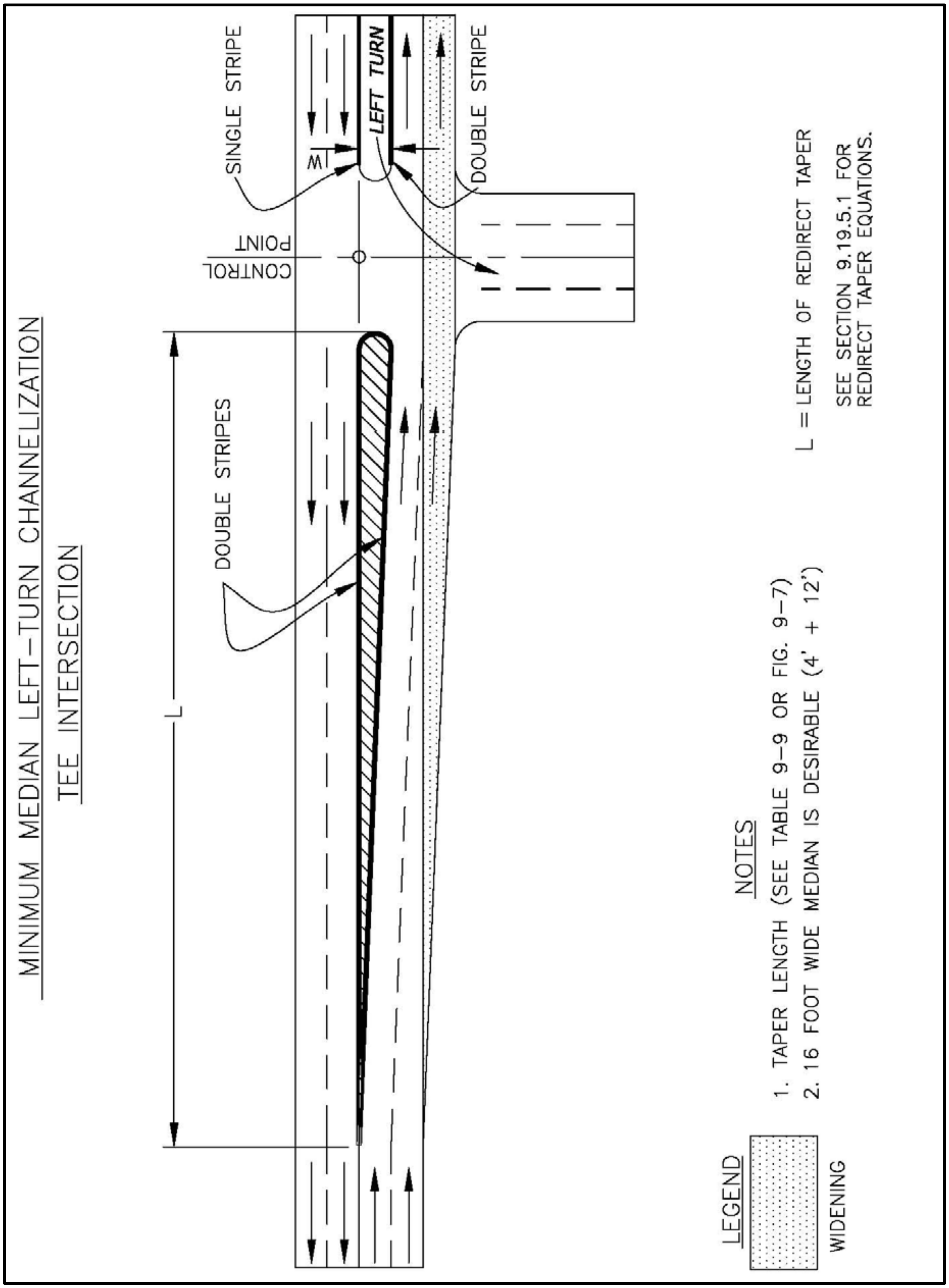


Figure 9-10B Minimum Median Left-Turn Channelization, Tee Intersection

Analysis of left-turn involved conflicts shows why their treatment is so critical. Left-turning vehicles conflict with:

- Opposing through traffic
- Crossing traffic
- Through traffic in the same direction

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

9.17.8.1 Median Left-Turn Lane Warrants

Because of the many variables involved, it is not feasible to develop guidelines for all conditions at signalized intersections. However, the following information should be considered in evaluating left-turn needs at specific locations.

At high-speed, rural-signalized intersections, separate left-turn lanes are considered necessary for safe operations. While capacity is not generally a problem, protection of queued left-turning vehicles from through vehicles is critical. Because the availability and cost of right of way are usually not a problem, separate left-turn lanes can, in most cases, be easily provided.

To facilitate flow where the intersection is unsignalized, the following guidelines are suggested:

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

9.17.9 Median Double Left and Triple Left Turn Lanes

Double left turns have been applied successfully nationwide at locations with severe capacity or operational problems. Their applicability is generally greatest at high volume intersections with significant left turning volumes in one or more directions. Double left-turn lanes should be considered at any signalized intersection with high design hour demand for left turns. As a general rule, left turn volumes of 300 vehicles per hour or more are appropriate for consideration for double left-turn lanes.

Left turning vehicles leave the through pavement to enter the median lanes in single file, but once within it, store in two lanes and, on receiving the green indication, turn simultaneously from both lanes. With three-phase signal control, such an arrangement results in an increase in capacity of approximately 180 percent of that of a single median lane. Because of the high turning volumes, double turning lanes should only be used with fully protected signal phasing.

Where turning lanes are designated for two-lane operation, the storage length is reduced to approximately 0.6 of that required for single-lane operation.

The widening on the curve for the two lanes of turning traffic is an important design element. Drivers are most comfortable with extra space between the turning queues of traffic. Because of off-tracking characteristics of vehicles and the relative difficulty of two abreast turns, a 36-foot width for the two lanes on the curve is desirable. In constrained situations, a 30-foot width on the curve is an acceptable minimum.

A summary of the current use of triple left turn lanes can be found in the Florida DOT report *Triple Left Turn Lanes at Signalized Intersections* **(6)**.

9.17.10 Median Lane Width

A median width of 20 feet or more is desirable at intersections with single-median lanes, but widths of 16 to 18 feet permit reasonably adequate arrangements. The minimum narrowed median width of no less than 4 feet is recommended, but 6 to 8 feet wide is preferred. These dimensions can be provided within a median 16 to 18 feet wide and a turning lane width of 10 to 12 feet. Widening of the highway may be required to accommodate the median area, equally on both sides of the centerline. Figures 9-11A and 9-11B show a minimum design for a median left turn lane within a 14 to 18 foot median. The left turn lane is 10 to 12 feet wide with a 4-foot wide median. Figure 9-11C below shows a design for a median left turn lane with a median greater than 18 feet.

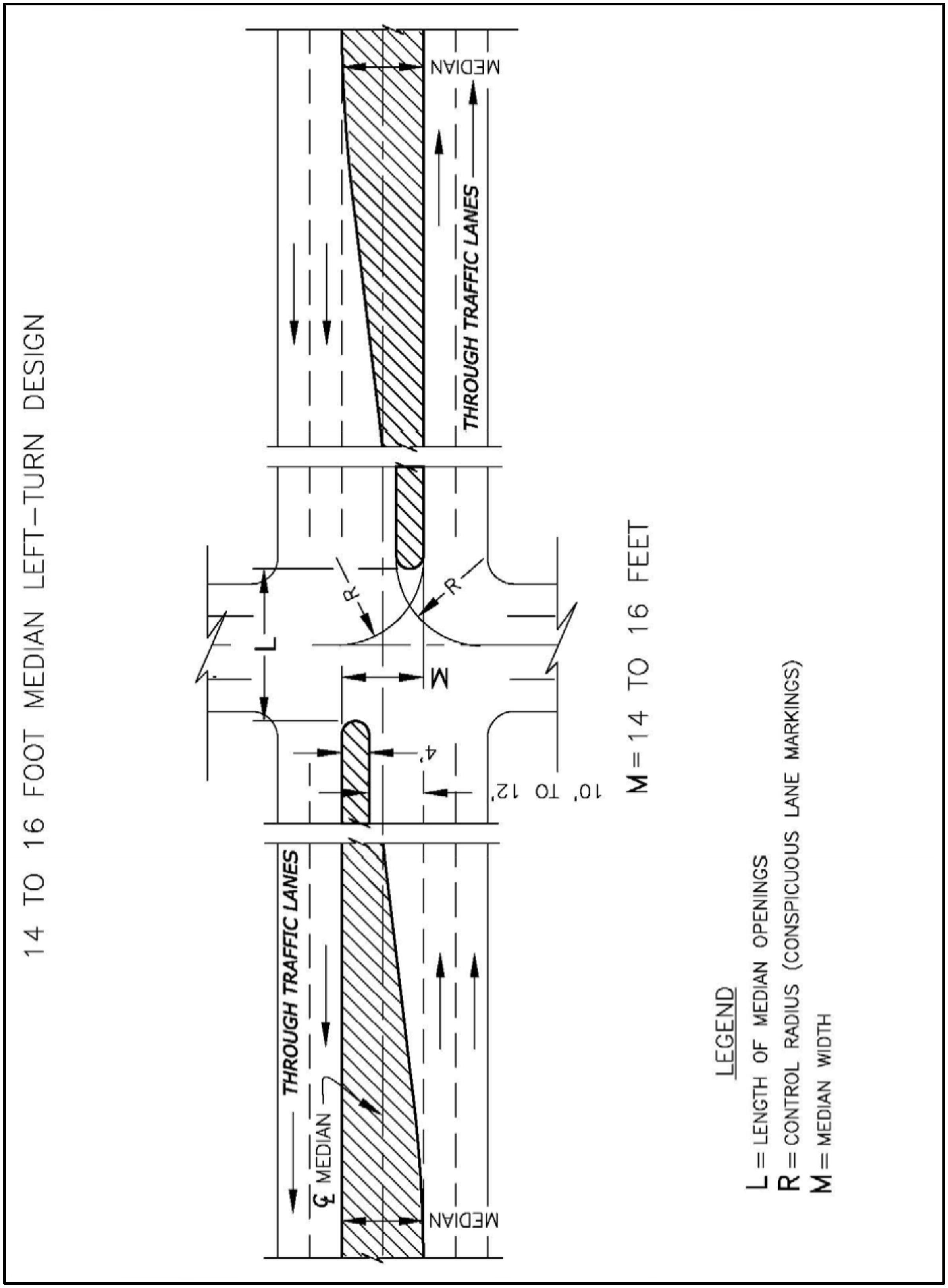


Figure 9-11A [Figure 9-50 of the *PGDHS* (1)] 14 to 16-Foot Median Left-Turn Design

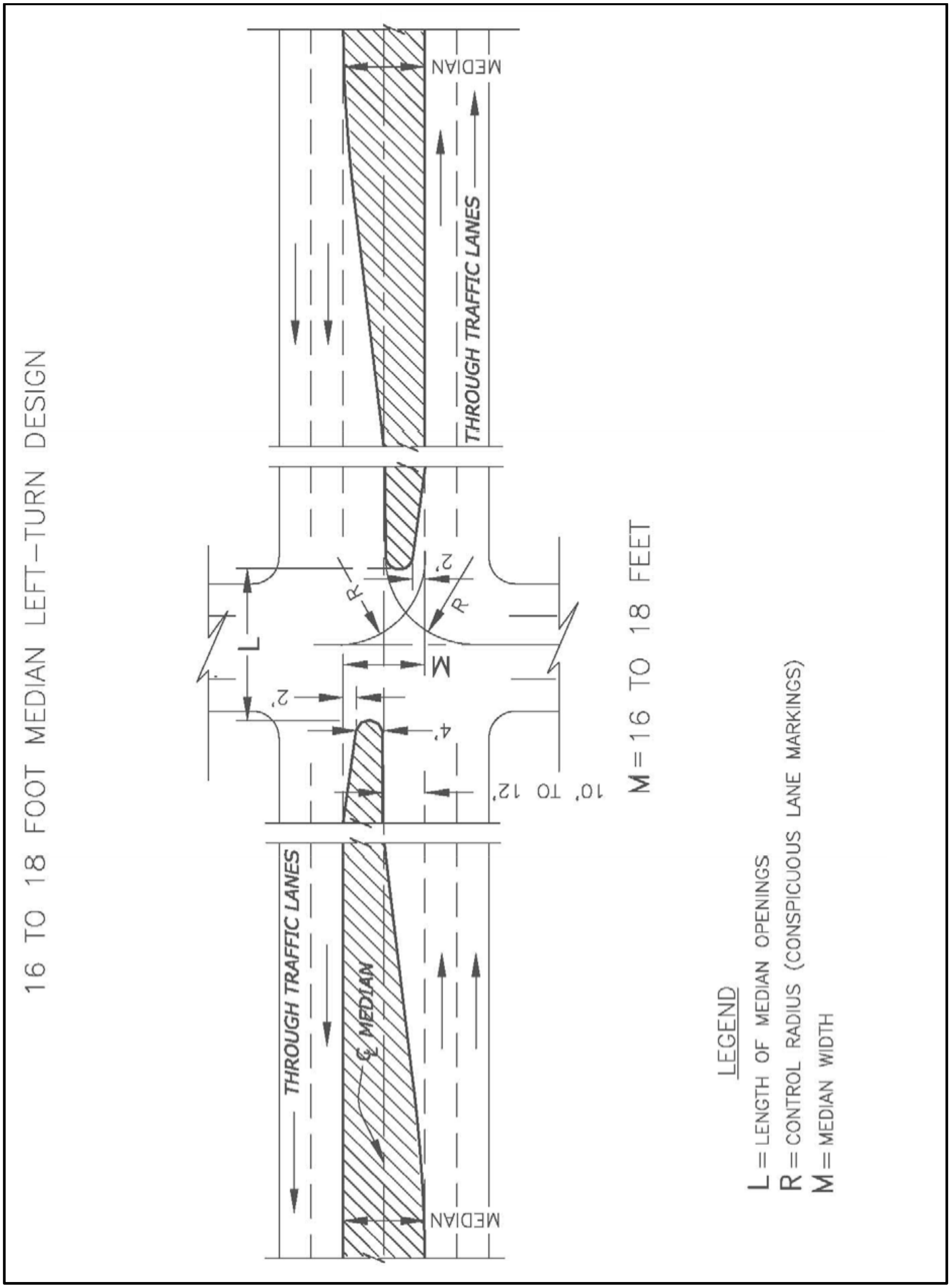


Figure 9-11B [Figure 9-50 of the *PGDHS* (1)] 16 to 18-Foot Median Left-Turn Design

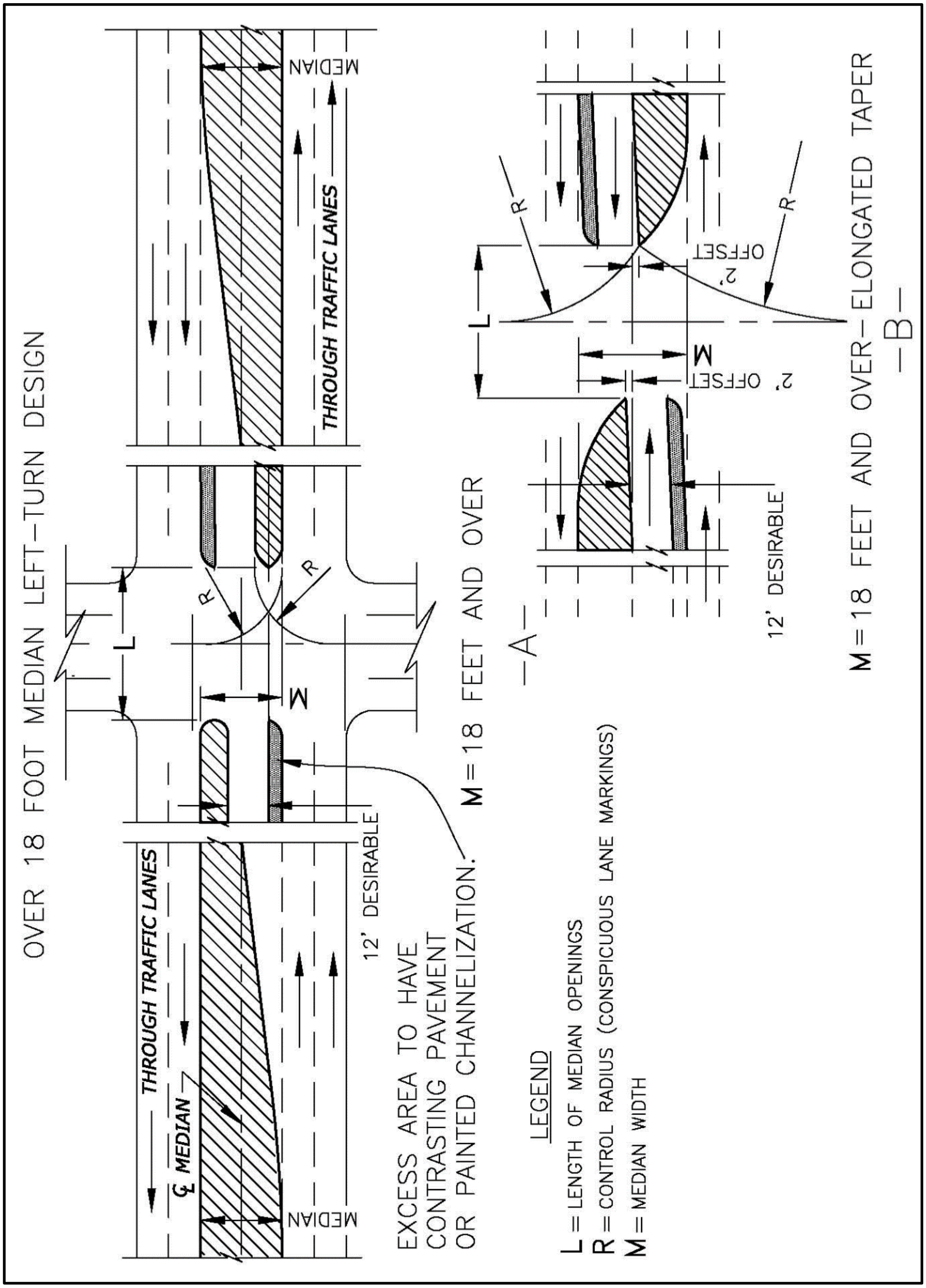


Figure 9-11C [Figure 9-51 of the *PGDHS* (1)] Median Left Turn Design for Median Width Greater Than 18 Feet

9.17.11 Median End Treatment

The form of treatment given the end of the narrowed median adjacent to lanes of opposing traffic depends largely on the available width. The narrowed median may be curbed to delineate the lane edge, to separate opposing movements, to provide space for necessary signs, markers, and lighting standards, and to protect pedestrians. To serve these purposes satisfactorily, a minimum narrowed median width of no less than 4 feet is recommended, and a median 6 to 8 feet wide is preferred. These dimensions can be provided within a median 16 to 18 feet wide and a turning lane width of 12 feet.

For medians wider than about 18 feet as shown in Figure 9-10C and in Figure 9-51 in the *PGDHS* **(1)**, it is usually preferable to align the left lane in a manner that will reduce the width of the divider to 6 to 8 feet immediately in advance of the intersection, rather than to align it exactly parallel with and adjacent to the through lane. This alignment will place the vehicle waiting to make the turn as far to the left as practical and thus provide appropriate visibility of opposing through traffic. The advantages of offsetting the left-turn lanes are:

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

For curbed dividers 4 feet or more in width at the narrowest end, the curbed nose can be offset from the opposing through traffic lane 2 feet or more, with gradual taper beyond to make it less vulnerable to contact by through traffic. The shape of the nose for curbed dividers 4 feet wide usually is semicircular, but for wider widths the ends are normally shaped to a bullet nose pattern to conform better to the paths of turning vehicles.

9.18 RIGHT-TURN LANES

9.18.1 General

Separate right-turning lanes shall be used for minor intersections that are skewed and all major intersections. Design of right-turn lanes is similar to that of left-turn lanes. A right-turn lane can fulfill one or more of the following functions:

- A means of safe deceleration outside the high-speed through lanes for right turning traffic.
- A storage area for right-turning vehicles to assist in optimization of traffic signal phasing.
- A means of separating right-turning vehicles from other traffic at stop controlled intersection approaches.

9.18.2 Tapers

Refer to section 9.17.7.

9.18.3 Storage

Design for storage at signalized intersections is based on arrival rates for right-turn volumes and departure conditions (i.e., available green time, cycle length). In designing for storage, the adjacent through-lane volume will often control the desirable length because:

- Right-turn lanes have greater capacity due to greater signal timing flexibility.
- There is potential for right turn on red movements.

For further information see the *Highway Capacity Manual* **(3)**.

9.18.4 Length

Right-turn lanes at stopped approaches should be of sufficient length to enable right-turning vehicles to bypass queued through and/or left turning vehicles. This allows the higher capacity right-turn movement to operate independently of other stopped movements.

The required length for a right-turn lane is calculated in the same manner as described in section 9.17.1. Signal timing, pedestrian activity, and vehicle arrival patterns are the most important aspects for consideration when designing the length. Normally, a minimum storage length of 100 feet should be provided in addition to the taper.

9.18.5 Width

Lane width requirements for right-turn lanes are similar to those for other lanes. In general, 12-foot lanes are desirable, although widths as low as 10 feet have been used in severely constrained situations unless large trucks and buses are using the lane.

The width of a separate right-turning lane shall normally provide at least one-way one-lane operation with passing permitted. In some cases, it may be necessary to provide one-way two- lane operation; additional shoulder width for emergency parking under this condition is usually not required. When two-lane operation is required, the maximum desirable turning radius shall be 200 feet.

Consider operational effects of barrier curbs on drivers. Right-turn lanes adjacent to such curbs should be designed to full widths to negate the constricting effects of the curb. This is particularly important if the gutter width dimension is nominal. An additional factor in establishing a right-turn lane is consideration of the location of bus stops. It may be necessary, e.g., to relocate a bus stop to midblock or to the far side if a right-turn lane is introduced.

9.18.6 Shoulders

See subsection 9.5.1.1.

9.19 INTERSECTION DESIGN ELEMENTS WITH FRONTAGE ROADS

Frontage roads are generally required contiguous to arterials or freeways where adjacent property owners are not permitted direct access to the major facility. Short lengths of frontage roads may be desirable along urban arterials to preserve the capacity and safety of the arterial through control of access. Much of the improvement in capacity and safety may be offset by the added hazard introduced where the frontage road and arterial intersect the at-grade crossroad. The added hazard results in part from the increase in the number of conflicting movements and from the confusing pattern of roadways and separations, which lead to wrong-way entry. Inevitably, where an arterial is flanked by frontage roads, the problems of design and traffic control at intersections are far more complex than where the arterial consists of a single roadway. Three intersections (two, if there is only one frontage road) actually exist at each cross street.

For satisfactory operation with moderate-to-heavy traffic volumes on the frontage roads, the outer separation should preferably be 150 feet or more in width at the intersection. A thorough discussion of the separation of main line and frontage road is discussed in Chapter 9 of the *PGDHS* **(1)**.

9.20 BICYCLES AT INTERSECTIONS

When on-street bicycle lanes and/or off-street bicycle paths enter an intersection, the design of the intersection should be modified accordingly. Further Guidance in providing for bicycles at intersections can be found in the AASHTO *Guide for Development of Bicycle Facilities* **(7).**

9.21 ADA RAMPS AT INTERSECTIONS

When designing a project that requires curbs and adjacent sidewalks to accommodate pedestrian traffic, proper attention should be given to the requirements of persons with mobility impairments who depend on wheelchairs and other mobility devices. See also Chapter 12.

9.22 LIGHTING AT INTERSECTIONS

Lighting may affect the safety of highway and street intersections as well as efficiency of traffic operations. Statistics indicate that the non-daylight accident rate is higher than that during daylight hours. This fact, to a large degree, may be attributed to impaired visibility. In urban and suburban areas where there are concentrations of pedestrians and roadside and intersectional interferences, fixed-source lighting tends to reduce accidents. The need for lighting of rural at- grade intersections depends on the planned geometrics and the turning volumes. Intersections that generally do not require channelization are seldom lighted. However, for the benefit of non-local highway users, lighting at rural intersections is desirable to aid the driver in ascertaining sign messages during non-daylight hours. See section 3.7.

Intersections with channelization, particularly multiple road geometrics, should include lighting. Large channelized intersections especially need illumination because of the higher range of turning radii that are not within the lateral range of vehicular headlight beams. Vehicles approaching the intersection also must reduce speed. The indication of this need should be definite and visible at a distance from the intersection that may be beyond the range of headlights. Illumination of at-grade intersections with fixed source lighting fulfills this need.

The planned location of intersection luminaire supports should be designed to present the least possible hazard to out-of-control vehicles. The breakaway support base should not be used within limits of an at-grade intersection, particularly in densely developed areas with adjacent sidewalks. When struck, these light standards could be a problem for pedestrians and compound damage to adjacent property and other vehicles. See the *AASHTO Roadside Design Guide* **(8)** for further design guidance.

9.23 DRIVEWAYS

Driveways are, in effect, at-grade intersections and should be designed consistent with the intended use. The number of accidents is disproportionately higher at driveways than at other intersections; thus, their design and location merit special consideration.

See Section 9.11.6 in the *PGDHS* **(1)**. The regulation and design of driveways are intimately linked with the right of way and zoning of the roadside. On new highways, the necessary right of way can be obtained to provide the desired degree of driveway regulation and control. In many cases, additional right of way can be acquired on existing highways or agreements can be made to improve existing undesirable access conditions. Often the desired degree of driveway control must be effected through the use of police powers to require permits for all new driveways and through adjustments of those in existence. Coordinate with the Region Access Unit. See CDOT *Standard Plans - M & S Standards* **(9)** and the *State Highway Access Code* **(4)** for design information on driveways.

9.24 RAILROAD-HIGHWAY GRADE CROSSINGS

A railroad-highway crossing, like any highway-highway intersection, involves either a separation of grades or a crossing at-grade. The horizontal and vertical geometrics of a highway approaching an at-grade railroad crossing should be constructed in a manner that does not divert a driver's attention from roadway conditions. Coordinate early and often with the Railroads. For further information on railroad crossings, refer to the *PGDHS* **(1)**.

REFERENCES

1. AASHTO. *A Policy on Geometric Design of Highways and Streets*, American Association of State Highway and Transportation Officials (AASHTO), Washington, D.C.: 2011
2. NCHRP. *NCHRP Report 279,* "Intersection Channelization Design Guide," Project Panel G3-30, National Research Council, National Academy Press, Washington, D.C., November 1985.
3. TRB. *Highway Capacity Manual*, Transportation Research Board, National Research Council, Washington, D.C.: 2010.
4 State Highway Access Code, 2 CCR 601-1, as adopted and amended by the Transportation Commission of Colorado, 1998 (with 2002 revisions).
5 U.S. Department of Transportation, Federal Highway Administration, *Manual on Uniform Traffic Control Devices for Streets and Highways (MUTCD),* Washington, D.C.: 2009.
6. FDOT. *Triple Left Turn Lanes at Signalized Intersections.* Florida Department of Transportation, 2002.
7. AASHTO. *Guide for the Development of Bicycle Facilities*, American Association of State Highway and Transportation Officials, Washington. D.C.: 2012.
8 AASHTO. *AASHTO Roadside Design Guide.* American Association of State Highway and Transportation Officials, Washington, D.C.: 2011.
9. CDOT. *CDOT Standard Plans - M & S Standards,* Colorado Department of Transportation, 2012.

CHAPTER 10
GRADE SEPARATIONS AND INTERCHANGES

10.0 INTRODUCTION AND GENERAL TYPES OF INTERCHANGES

The ability to accommodate high volumes of traffic safely and efficiently through intersections depends largely on the arrangement that is provided for handling intersecting traffic. The greatest efficiency, safety, and capacity, and least amount of air pollution are attained when the intersecting through traffic lanes are grade separated. An interchange is a system of interconnecting roadways in conjunction with one or more grade separations that provide for the movement of traffic between two or more roadways or highways on different levels.

Interchange design is the most specialized and highly developed form of intersection design. The designer should be thoroughly familiar with the material in Chapter 9 before starting the design of an interchange. Relevant portions of the following material covered in Chapter 9 also apply to interchange design:

- general factors affecting design
- basic data required
- principles of channelization
- design procedure
- design standards

Material previously covered is not repeated. The discussion which follows covers modifications in the above-mentioned material and additional material pertaining exclusively to interchanges.

The economic effect on abutting properties resulting from the design of an intersection at-grade is usually confined to the area in the immediate vicinity of the intersection. An interchange or series of interchanges on a freeway or expressway through a community may affect large contiguous areas or even the entire community. For this reason, consideration should be given to an active public process to encourage context sensitive solutions. Interchanges must be located and designed to provide the most desirable overall plan of access, traffic service, and community development.

The type of grade separation and interchange, along with its design, is influenced by many factors such as highway classification, character and composition of traffic, design speed, and degree of access control. These controls plus signing requirements, economics, terrain, environment, and right-of-way are of great importance in designing facilities with adequate capacity to safely accommodate the traffic demands. Interchange types are characterized by the basic shapes of ramps, namely, diamond, loop, directional, "urban" and cloverleaf interchanges. Figures 10-1A, B, C, D and E illustrate these basic interchange types. These examples can further be classified as either local street interchanges or freeway-to-freeway interchanges.

Although each interchange presents an individual challenge, it must also be considered in conjunction with adjacent interchanges, driver expectancy, and at-grade intersections in the corridor as a whole. For further information, see Chapter 10 of the *PGDHS* **(1)**. A more detailed description of all the basic types is found later in this chapter.

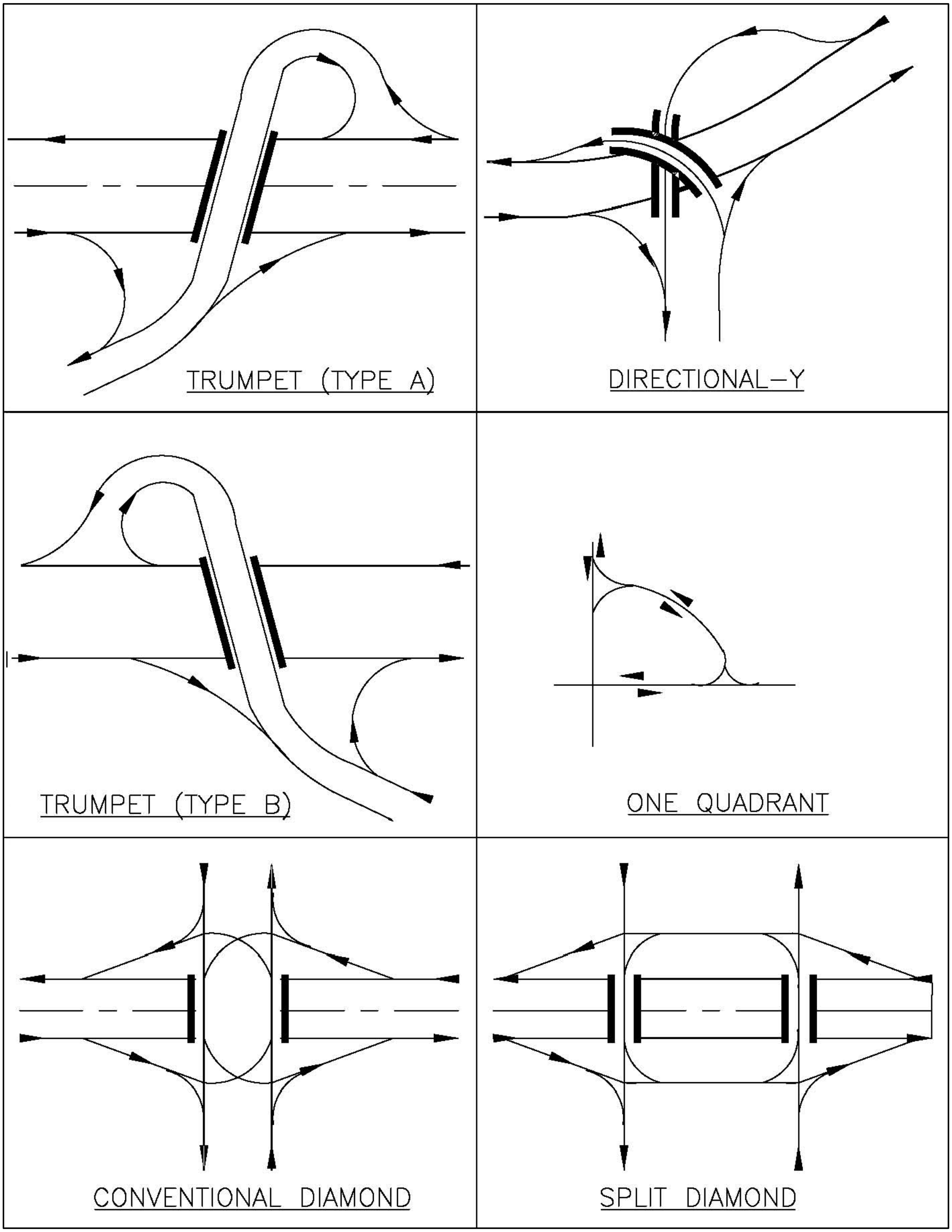


Figure 10-1A Interchange Types

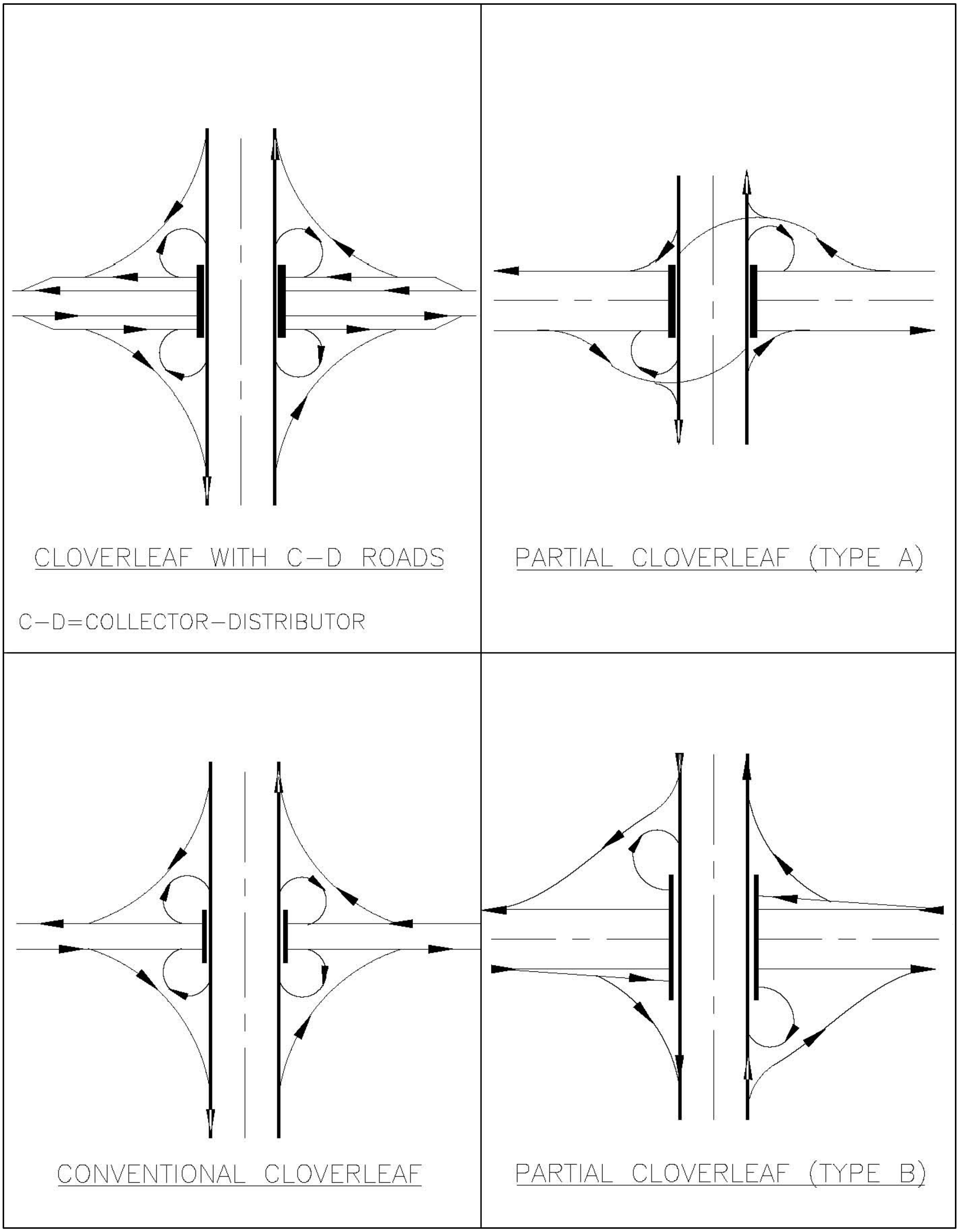


Figure 10-1B Interchange Types

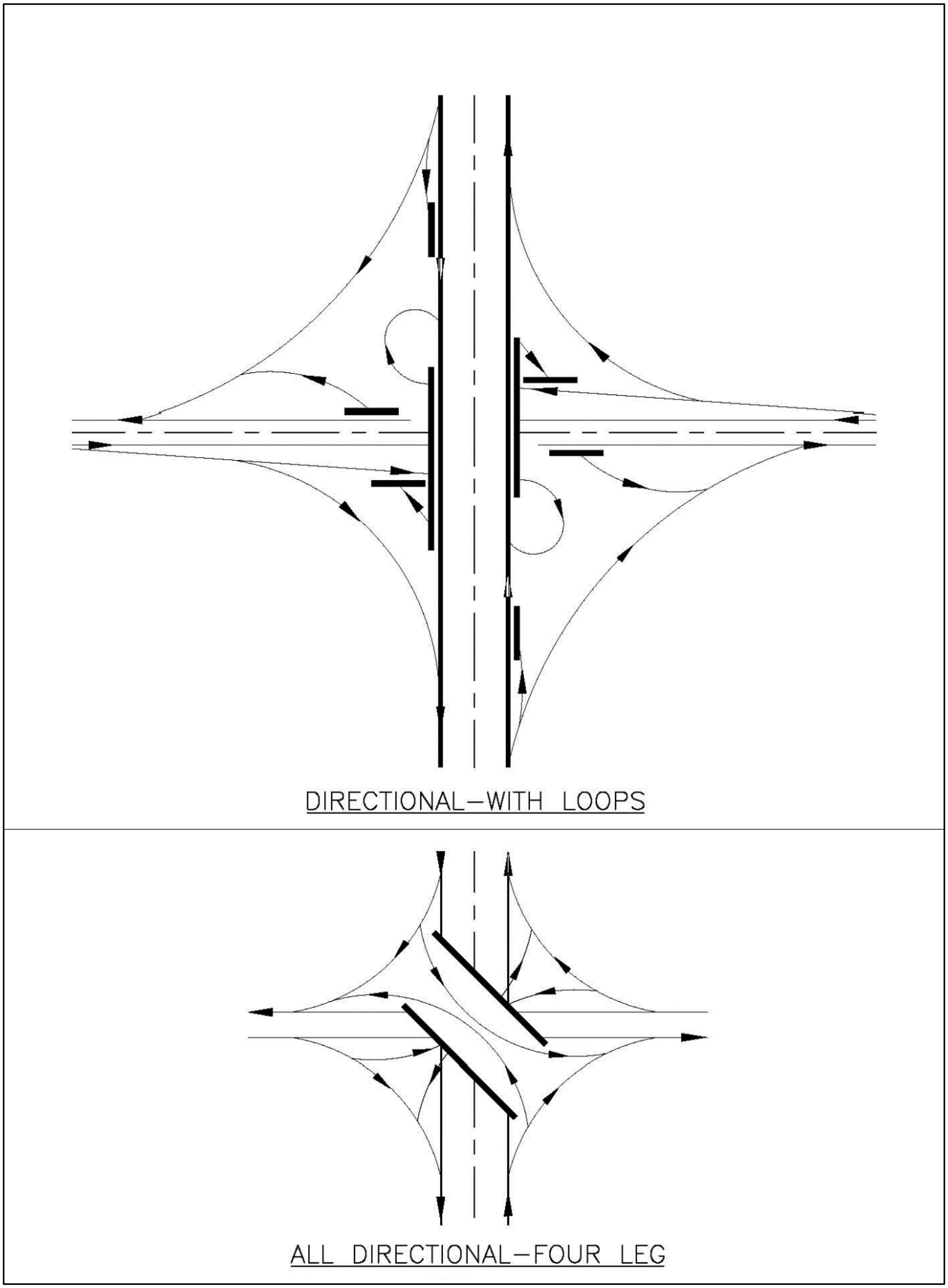


Figure 10-1C Interchange Types

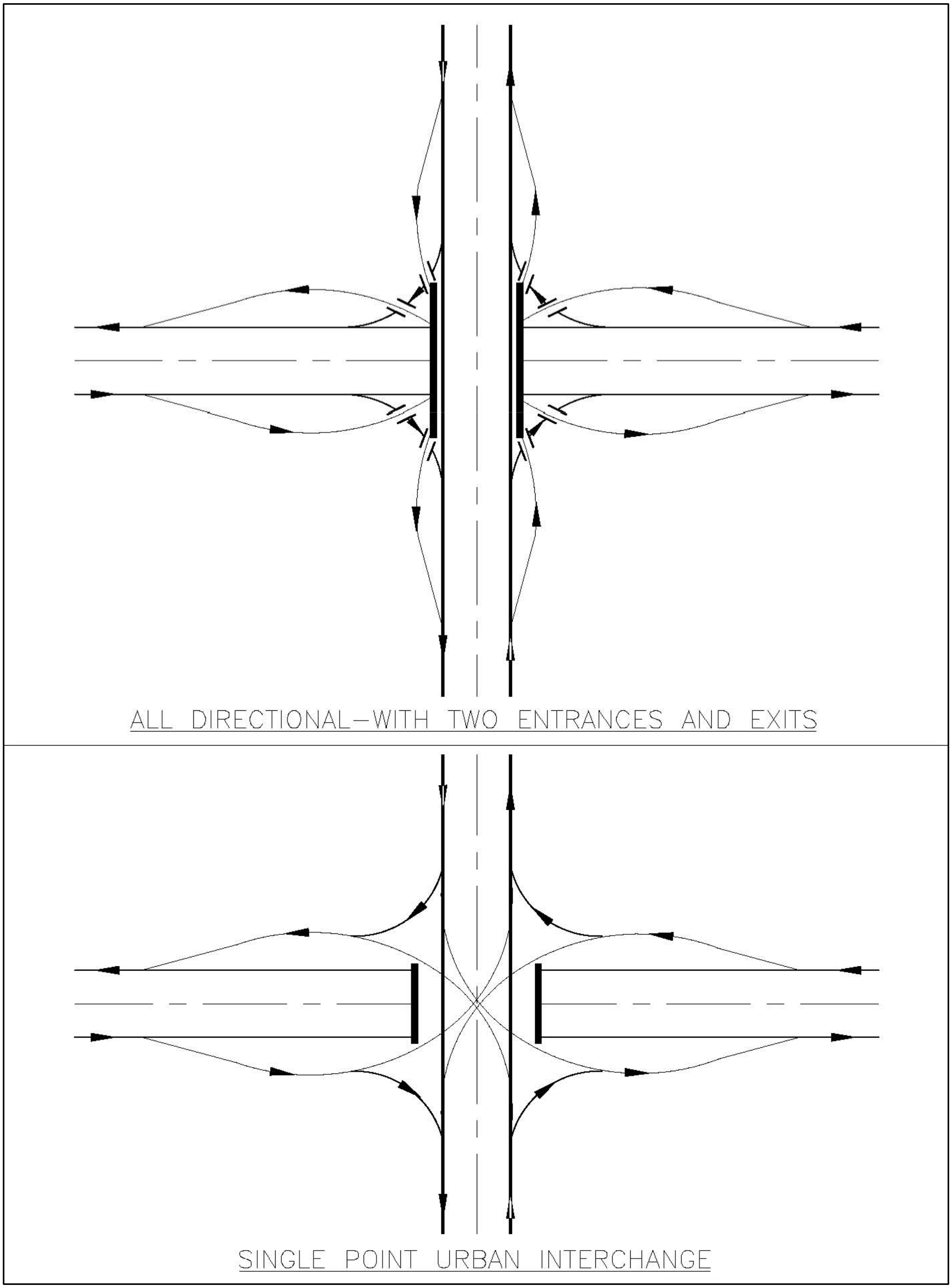


Figure 10-1D Interchange Types

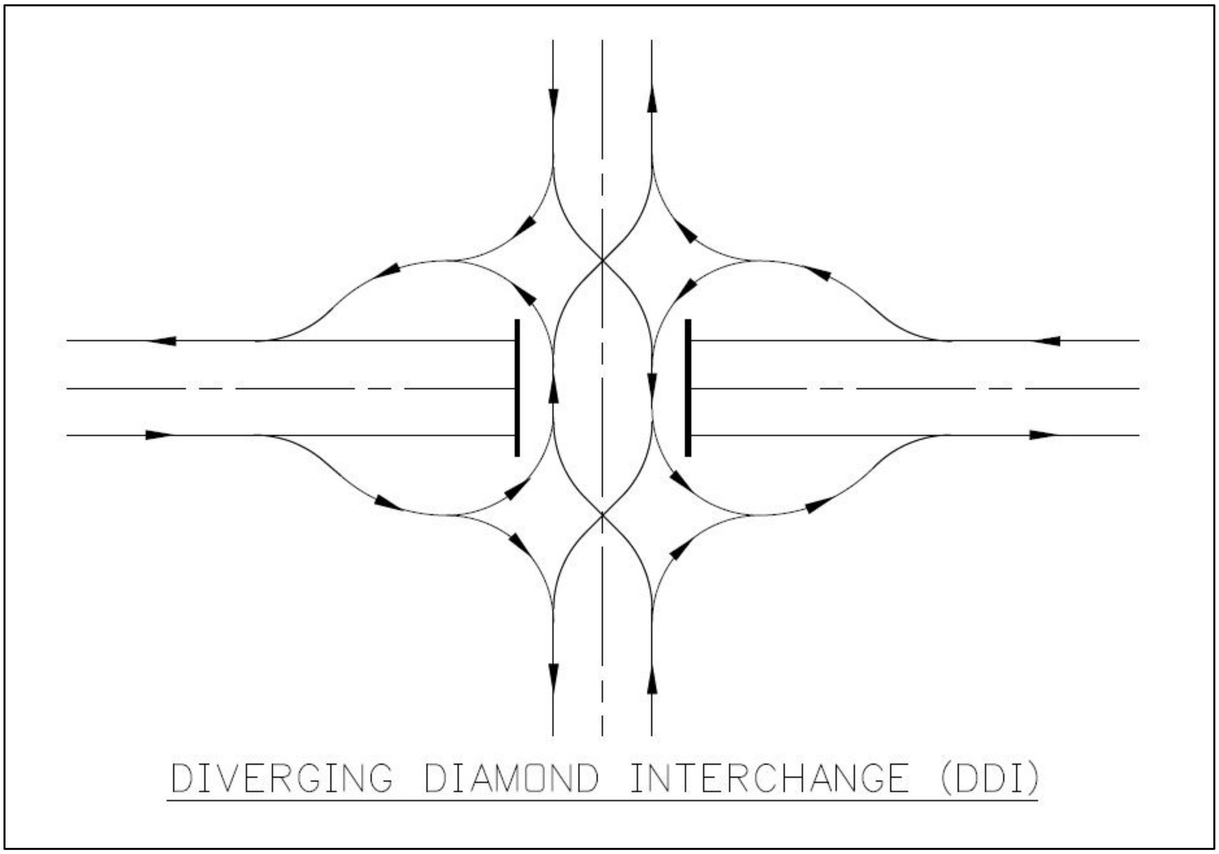


Figure 10-1E Interchange Types

10.1 WARRANTS FOR INTERCHANGES AND GRADE SEPARATIONS

10.1.1 Interchange and Grade Separation Warrants

All connections to freeways are by traffic interchanges. An interchange or separation may be warranted as part of an expressway (or in special cases at the junction of two non-access controlled highways). Because of the wide variety of site conditions, traffic volumes, highway types, and interchange layouts, the warrants that justify an interchange may differ at each location. Warrants, therefore, are necessarily general and must be based on engineering judgment. CDOT *Policy Directive 1601.0* **(2)** must be followed. When determining conditions that may warrant an interchange, the following should be considered:

- Design designation
- Reduction of bottlenecks or spot congestion
- Reduction of crash frequency and severity – Some at-grade intersections have a disproportionate frequency of serious crashes. If inexpensive methods of reducing crashes are likely to be ineffective or impractical, a highway grade separation or interchange may be warranted. Higher crash frequencies are often found at intersections between comparatively lightly-traveled highways in sparsely settled rural areas where speeds are high. Serious crashes at heavily traveled intersections may also warrant interchange facilities. In addition to the

reduction in crash frequency and severity, the operational efficiency for all traffic movements is also improved with interchanges.
- Site topography - The site topography and the grades of the intersecting roadways are important to determine interchange type and location. The right-of -way required for an interchange is dependent largely on the type of highway, topography, and the overall type of interchange.
- Road-user benefits
- Traffic volume warrant - Except on freeways, interchanges usually are provided only where crossing and turning traffic cannot readily be accommodated on a less costly at-grade intersection.
- Transit
- Functional classification of the road

10.2 ADAPTABILITY OF HIGHWAY GRADE SEPARATIONS AND INTER-CHANGES

The three types of intersections are:

- at-grade intersections
- highway grade separations without ramps
- interchanges

Factors that would determine the need for an interchange and its type:

- Traffic and Operation
- Site Conditions
- Type of Highway - The hazard from stopping and direct turns at an intersection increases with the design speed so that high-design-speed highways warrant interchange treatment earlier than low-design-speed roads with similar traffic volumes.
- Intersecting Facility - The extent or degree to which local service must be maintained or provided also is of concern in the selection of the type of intersection. Local service can be provided readily on certain types of at-grade intersections, whereas considerable additional facilities may be necessary on some types of interchanges.
- Safety
- Stage Development - Where the ultimate development consists of a single grade-separation structure, stage construction may not be economical unless provisions are made in the original design for a future stage of construction. Ramps, however, are well adapted to stage development.
- Economics - Initial cost needs to be considered. The interchange is the costliest type of intersection because of the cost of the structure, ramps, through roadways, grading and landscaping of large areas.
- Maintenance costs may be a factor in the type of intersection. Interchanges have large pavement and variable slope areas, the maintenance of which, together with that of the structure, signs, and landscaping, exceeds that of an at-grade intersection.

In a complete analysis of the adaptability of interchanges, it is necessary to compare vehicular operating costs of all traffic with those for other intersections.

10.3 GRADE SEPARATION STRUCTURES

In any single separation structure, care should be exercised in maintaining a constant clear roadway width and a uniform protective railing or parapet.

The type of structure best suited to grade separations is one that gives drivers little sense of restriction. Where drivers take practically no notice of a structure over which they are crossing, sudden and erratic changes in speed and/or direction are unlikely. On the other hand, it is virtually impossible not to notice a structure overpassing the roadway being used. For this reason, every effort should be made to design the structure so that it fits the environment in a pleasing and functional manner without drawing unnecessary or distracting attention.

A detailed study should be made at each proposed highway grade separation to determine whether the main road should be carried over or under the structure. Often the choice is dictated by features such as cost, environmental impacts, topography, or highway classification. It may be necessary to make several nearly complete preliminary layout plans before a decision regarding the most desirable general layout plan can be reached.

As a rule, a design that best fits the existing topography is the most economical to construct and maintain, and this factor becomes the first consideration in design.

The clear width on bridges should be as wide as the approach pavement including shoulders, in order to give the driver a secure feeling. When the full approach roadway is continued across the structure, the parapet rail, both left and right, should align with the guardrail on the approach roadway.

Minimum lateral clearances at underpasses and retaining walls should include any provisions for the dynamic lateral deflection that the guardrail may require.

Additional information on vertical clearances is in section 3.3.

For more information on grade separation structures, see Chapter 10 of the *PGDHS* **(1)**.

10.4 INTERCHANGES

10.4.1 General

There are several basic interchange forms or geometric patterns of ramps for turning movements at a grade separation. Their application at a particular site is determined by the number of intersection legs, the expected volumes of through and turning movements, topography, culture, design controls, proper signing, and the designer's judgment.

The design and selection of an interchange type are influenced by many factors as described elsewhere in this chapter. Even though interchanges are, of necessity, designed to fit specific conditions and controls, the pattern of interchange ramps along a freeway should follow some degree of consistency. From the standpoint of driver expectancy, all interchanges should have one point of exit located in advance of the crossroad wherever practical. It is desirable to rearrange portions of the local street system in conjunction with freeway construction in order to achieve an effective overall plan of traffic service and community development.

Signing and operations are major considerations in the design of interchanges. Each design must be tested to determine if it can be signed properly for the smooth, safe flow of traffic. The need to simplify interchange design from the standpoint of signing and driver comprehension cannot be overstated.

From the standpoint of safety and in particular to prevent wrong-way movements, all freeway interchanges with non-access controlled highways should provide ramps to serve all basic directions. Drivers expect freeway-to-freeway interchanges to provide all directional movements. As a special case treatment, a specific freeway-to-freeway movement may be omitted if the turning traffic is minor and can be accommodated and given the same route signing via other nearby major state highways or other freeway facilities.

The basic interchange configurations are:

- Three-leg designs
- Four-leg designs
    - Ramps in one quadrant
    - Diamond interchanges
    - Diverging diamond interchanges (DDI)
    - Single-point urban interchanges (SPUI)
    - Cloverleafs
    - Partial cloverleaf ramp arrangements
    - Directional and semidirectional interchanges
- Other interchange configurations
    - Offset Interchanges
    - Combination Interchanges

CDOT and FHWA discourage the creation of partial interchanges and these should be avoided. See Chapter 10 of the *PGDHS* **(1).**

10.5 GENERAL DESIGN CONSIDERATIONS

Except for accident data, all basic data listed under Chapter 9 is also required for interchange design. This includes:

- Design Speed
- Design Traffic Volumes
- Levels of Service
- Pavement and Shoulders
- Curbs
- Superelevation
- Grades
- Structures
- Horizontal and Vertical Clearance Sight Distance

Data relative to community service (community access needs), traffic (projected traffic volumes), physical (topographic), environmental (NEPA considerations), economic factors (potential right-of-way acquisition), and potential area development which may affect design, should be obtained prior to interchange design (context sensitive solutions). Specifically, the following information should be available:

- The location and standards (types) of existing and proposed local streets and highway development including types of traffic (access) control.
- Present and potential traffic circulation over the affected local roads or streets.
- Existing and proposed land use including such developments as shopping centers, recreational facilities, housing developments, schools, churches, hospitals, and other institutions.
- A traffic flow diagram (a schematic interchange layout) showing annual average daily traffic and design hourly volumes, as well as time of day (a.m. or p.m.), anticipated on the freeway ramps and any affected local roads or streets.
- The relationship with (distances to and from) adjacent interchanges.
- The location of major utilities and multi-modal facilities (e.g., railroads, transit, airports).

10.5.1 Determination of Interchange Configuration

The need to use interchanges may occur in the design of roadways of all functional classifications, as discussed under section 10.1.

In rural areas, the problem of interchange-type selection is solved on the basis of service demand. The predominant rural interchange type in use in Colorado is the diamond interchange.

A combination of directional, semi-directional, and loop ramps may be appropriate where turning volumes are high for some movements and low for others. When loop ramps are used in combination with direct and semi-direct ramp designs, it is desirable that the loops be arranged so that weaving sections will not be created.

A cloverleaf interchange is the minimum design that can be used at the intersection of two fully controlled access facilities or where left turns at-grade are prohibited. A cloverleaf interchange is adaptable in a rural environment where right-of-way is not prohibitive and weaving is minimal. In the decision process to use cloverleaf interchanges, careful attention should be given to the potential improvement in operational quality that would be realized if the design included collector-distributor roads on the major roadway.

Simple diamond interchanges are the most common type of interchange for the intersection of a major roadway with a minor facility. The capacity of a diamond interchange is limited by the capacity of the at-grade terminals of the ramps at the cross road. High through and turning volumes could preclude the use of a simple diamond unless signalization is used.

Diverging diamond interchanges (DDI) are a variation of conventional diamond interchanges. The DDI uses directional crossover intersections to shift traffic on the cross street to the left-hand side between the ramp terminals within the interchange. Crossing the through movements to the opposite side replaces left-turn conflicts with same-direction merge/diverge movements and eliminates the need for exclusive left-turn signal phases to and from the ramp terminals. All connections from the ramps to and from the cross street are joined outside of the cross-over intersections, and these connections can be controlled by two-phase signals, have stop or yield control, or can be free flowing. In addition to the added safety benefits, DDI's typically have higher left-turn volume capacity and improved operations compared to conventional signalized diamond interchanges due to shorter cycle lengths, reduced time lost per cycle phase, reduced stops and delay, and shorter queue lengths.

Single-point urban interchanges (SPUI) is an interchange configuration that has all four turning movements controlled by a single traffic signal and the opposing left-turns operate to the left of each other, so their paths do not intersect. As a result, a major source of traffic conflict is eliminated, increasing the overall intersection efficiency and reducing the traffic signal phasing needed from four-phase to three-phase operation.

Partial cloverleaf designs may be appropriate where rights-of -way are not available in one or two quadrants or where one or two movements in the interchange are disproportionate to the others, especially when they require left turns across traffic. In the latter case, loop ramps may be utilized to accommodate the heavy left-turn volume.

Interchanges in rural areas are widely spaced and can be designed on an individual basis without any appreciable effect from other interchanges within the system.

The final configuration of an interchange may be determined by the need for route continuity, uniformity of exit patterns, single exits in advance of the separation structure, elimination of weaving on the main facility, signing potential, and available right-of-way.

Interchange-type determination in an urban environment requires considerable analysis of regional conditions so that the most practical interchange configurations can be developed.

10.5.2 Approaches to the Structure

See the *PGDHS* **(1)** Chapter 10.

10.5.2.1 Alignment, Profile, and Cross Section

Traffic passing through an interchange should be afforded the same degree of utility and safety as that given on the approaching highways. The design elements in the intersection area, therefore, should be consistent with those on the approaching highways, even though this may be difficult to attain. Preferably, the geometric design at the highway grade separation should be better than that for the approaching highways to counterbalance any possible sense of restriction caused by the structure. When it is practical to design only one of the intersecting roadways on a tangent with flat grades, it should be the major highway.

The general controls for horizontal and vertical alignment and their combination, as stated in Chapter 3, should be adhered to closely. Particular attention should be given to providing decision sight distance in situations where drivers must make complex or instantaneous decisions within interchanges.

The longitudinal distance needed for adequate design of a grade separation depends on the design speed, the roadway gradient, and the amount of rise or fall needed to achieve the separation. The amount of rise or fall needed will depend on the amount of vertical clearance needed in addition to the structure depth. The approximate distance needed to achieve a grade separation (assuming flat terrain) can be determined using Figure 10-8 from the *PGDHS* **(1)** Chapter 10. This table will provide an approximate distance to achieve grade separation; however, a detailed design profile should be developed to confirm all design criteria have been met.

Typically, a 20- to 22-foot difference in elevation is needed at a grade separation of two highways for essential vertical clearance and structure depth. The same dimension generally applies to a highway undercrossing a railroad, but about 28 feet is needed for a highway overcrossing of a main-line railroad.

10.5.2.2 Sight Distance

Sight distance on the highways through a grade separation should be at least as long as that needed for stopping, and preferably longer. Where exits are involved, decision sight distance is preferred, although not always practical. Ramp terminals at crossroads should be treated as at- grade intersections and should be designed in accordance with Chapter 9.

10.5.3 Interchange Spacing

In general, the minimum interchange spacing should be one mile in urban areas and two miles in rural areas. In urban areas, spacing of less than one mile may be allowed with the use of auxiliary lanes, grade-separated ramps, or collector-distributor roads.

10.5.4 Uniformity of Interchange Patterns

Left-entrances are undesirable due to difficulties merging with high-speed through traffic. Except in highly special cases, all entrance and exit ramps should be on the right. To the extent practical, all interchanges along the freeway should be reasonably uniform in geometric layout and general appearance.

10.5.5 Route Continuity

See the *PGDHS* **(1)** Chapter 10.

10.5.6 Coordination of Lane Balance and Basic Number of Lanes

Fundamental to establishing the number and arrangement of lanes on a freeway is the designation of the basic number of lanes. A certain consistency should be maintained in the number of lanes provided along any route of arterial character. Thus the basic number of lanes is defined as a minimum number of lanes designated and maintained over a significant length of a route, irrespective of changes in traffic volume and lane-balance needs. Stating it another way, the basic number of lanes is a constant number of lanes assigned to a route, exclusive of auxiliary lanes.

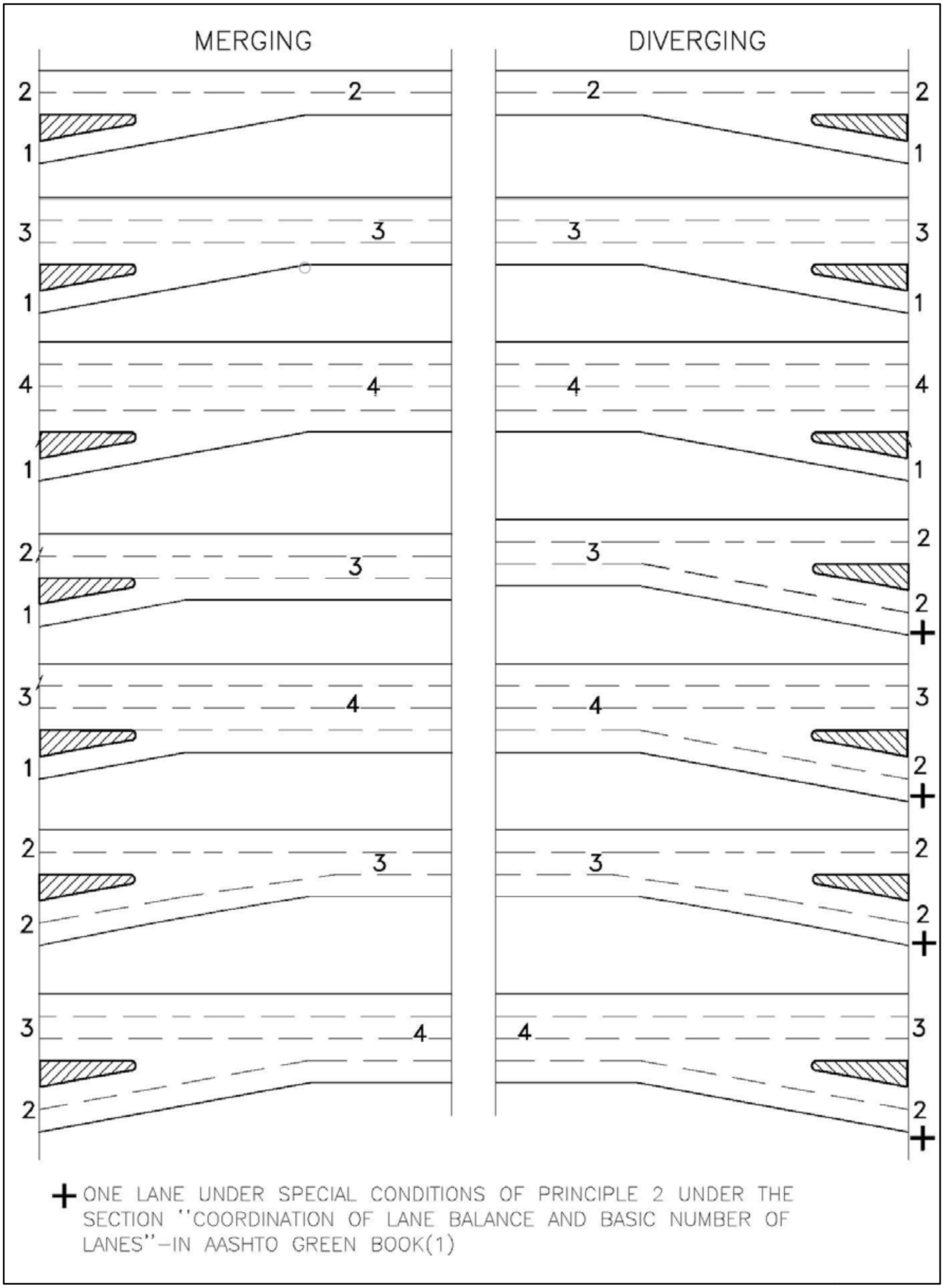


Figure 10-2 [Figure 10-50 of the *PGDHS* (1)] Typical Examples of Lane Balance

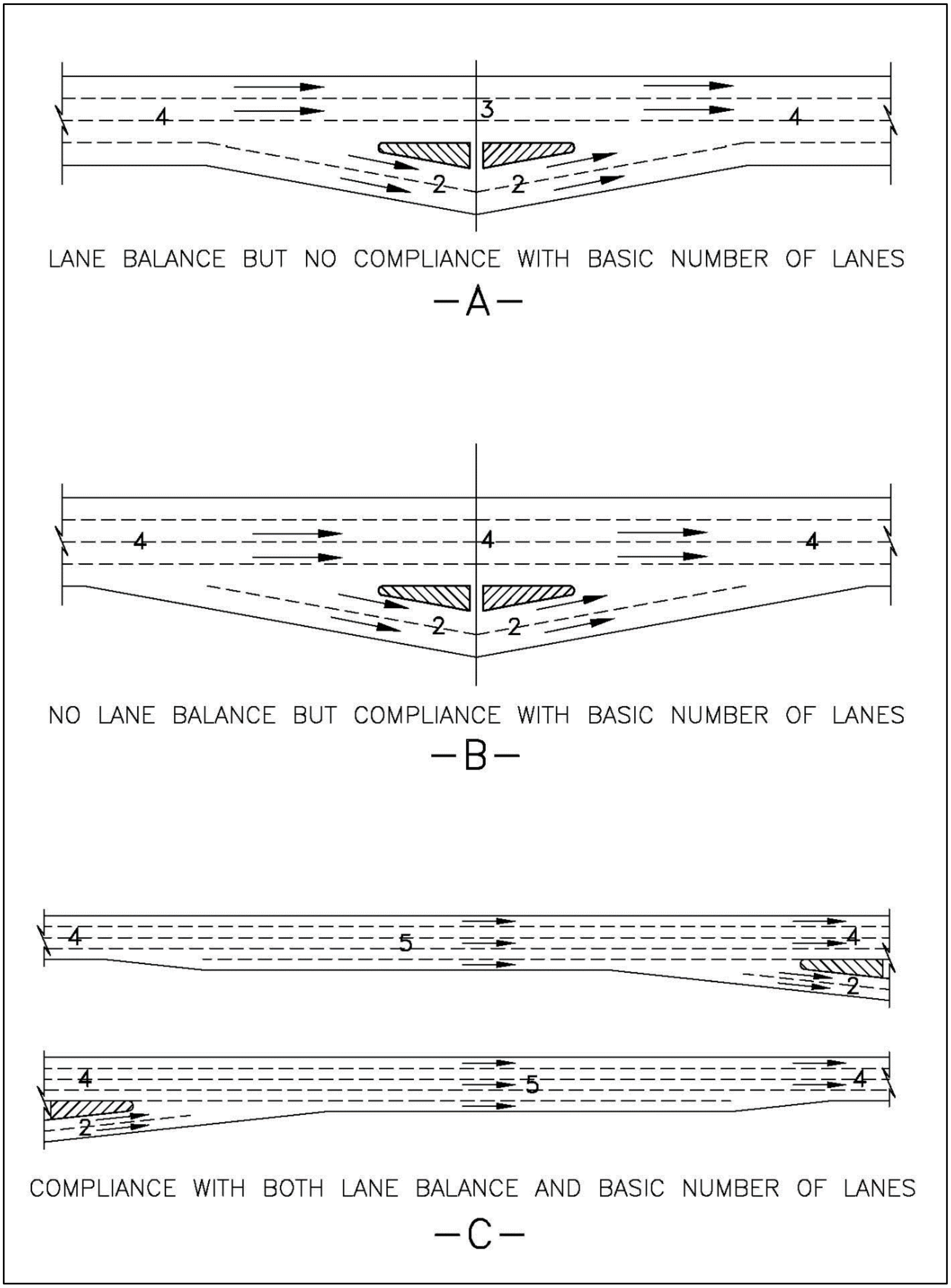


Figure 10-3 [Figure 10-51 of the *PGDHS* (1)] Coordination of Lane Balance and Basic Number of Lanes

10.5.7 Auxiliary Lanes

An auxiliary lane is defined as the portion of the roadway adjoining the traveled way for speed change, turning, storage for turning, weaving, truck climbing and other purposes supplementary to through-traffic movement. The width of an auxiliary lane should be equal to the through lanes. An auxiliary lane may be provided to comply with the concept of lane balance, to comply with capacity needs, or to accommodate speed changes, weaving, and maneuvering of entering and leaving traffic. Where auxiliary lanes are provided along freeway main lanes, the adjacent shoulder should desirably be 8 to 12 feet in width, with a minimum 6-foot wide shoulder considered.

10.5.8 Lane Reduction

If a basic lane or an auxiliary lane is to be dropped between interchanges, it should be accomplished at a distance of 2,000 to 3,000 feet from the previous interchange. The lane reduction should not be made so far downstream that motorists become accustomed to a number of lanes and are surprised by the reduction. The minimum taper rate should be 50:1, and the desirable taper rate is 70:1.

Left side lane reductions should be avoided because of generally higher speeds and less familiarity with left-side merges.

10.5.9 Weaving Sections

A weaving section is a length of one-way roadway where vehicles are crossing paths, changing lanes, or merging with through traffic as they enter or exit a freeway or collector-distributor road.

Weaving sections occur frequently along freeways and expressways in urban areas. Weaving sections are inherent to some interchanges, such as the cloverleaf and those with semi-direct connections. They are also found between ramps of closely spaced, successive interchanges.

Because considerable turbulence occurs throughout weaving sections, interchange designs that eliminate weaving or remove it from the main facility are desirable. Weaving sections may be eliminated from the main facility by the selection of interchange types that do not have weaving or by the incorporation of collector-distributor roads in the design.

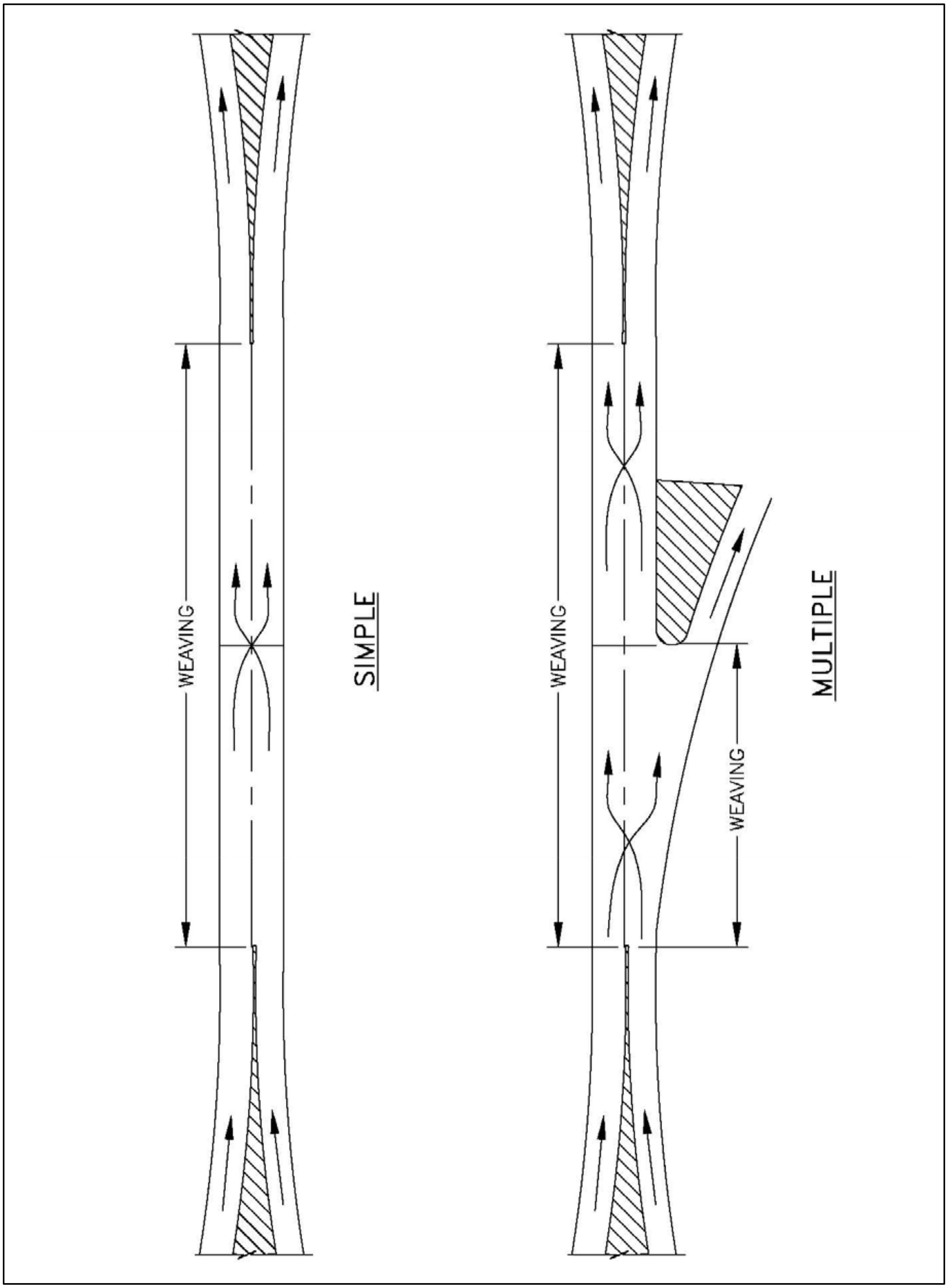


Figure 10-4A Types of Weaving Sections

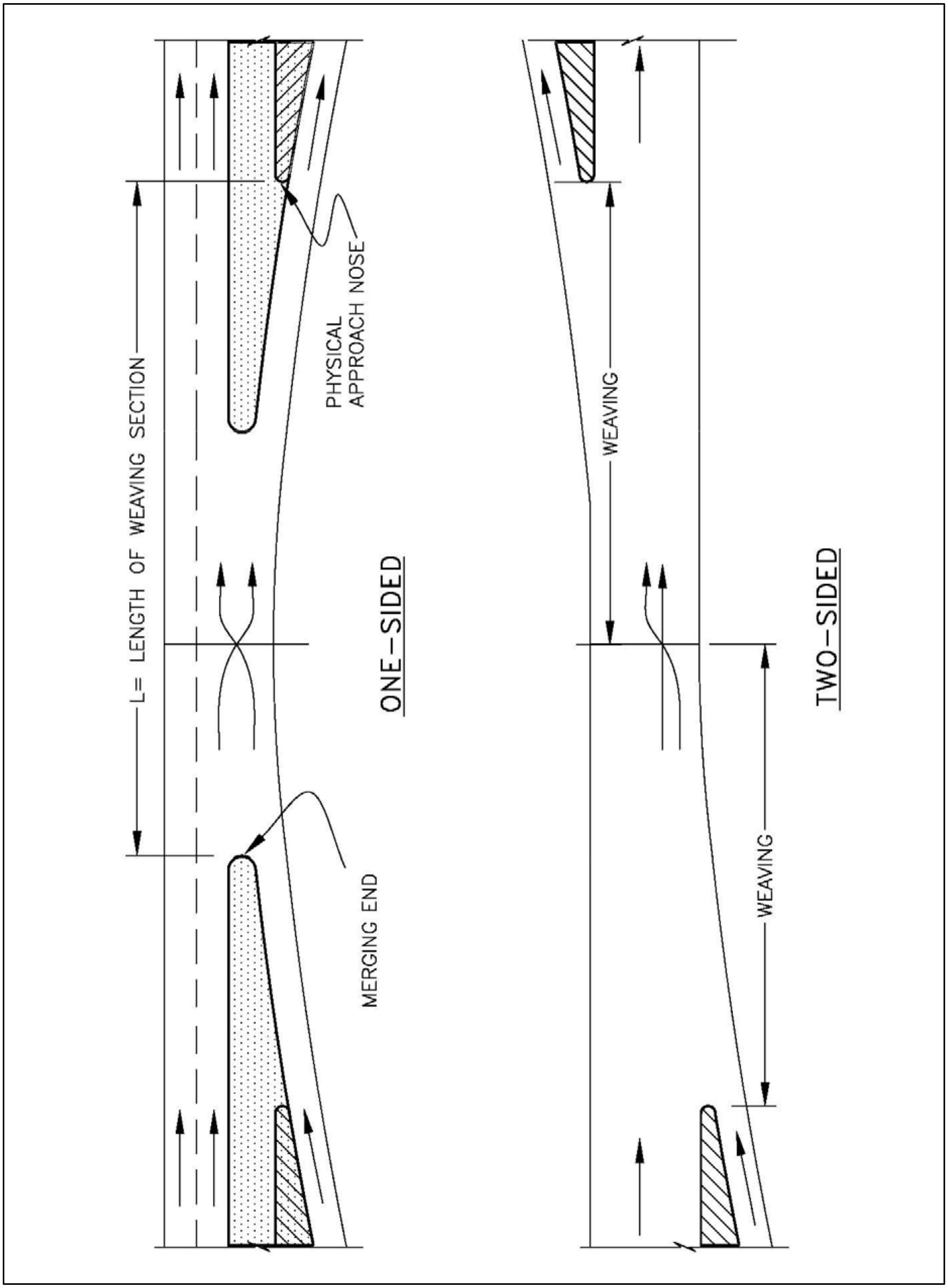


Figure 10-4B Types of Weaving Sections

A simple weaving section has an entrance at the upstream end and an exit at the downstream end. A multiple weaving section is characterized by more than one point of entry followed by one or more points of exit. For the various types of weaving situations, see Figures 10-4A and 10-4B.

The capacity of weaving sections may be seriously restricted unless adequate length and width are provided through the weaving section along with lane balance. See Chapter 2 of the *PGDHS* **(1)** for procedures for determining weaving lengths and widths.

Figures 10-5A and 10-5B give examples of balanced lane conditions. The established relation of factors used in the design of weaving sections is found in Chapter 13 of the Highway Capacity Manual (3). Weaving sections in urban areas should be designed for level of service C or D where possible. Weaving sections in rural areas should be designed for level of service B or C. Volume in equivalent passenger cars per hour (PCPH) is adjusted for freeway grade and truck volumes.

The Region Traffic and Safety Engineer should be consulted for difficult weaving analysis problems.

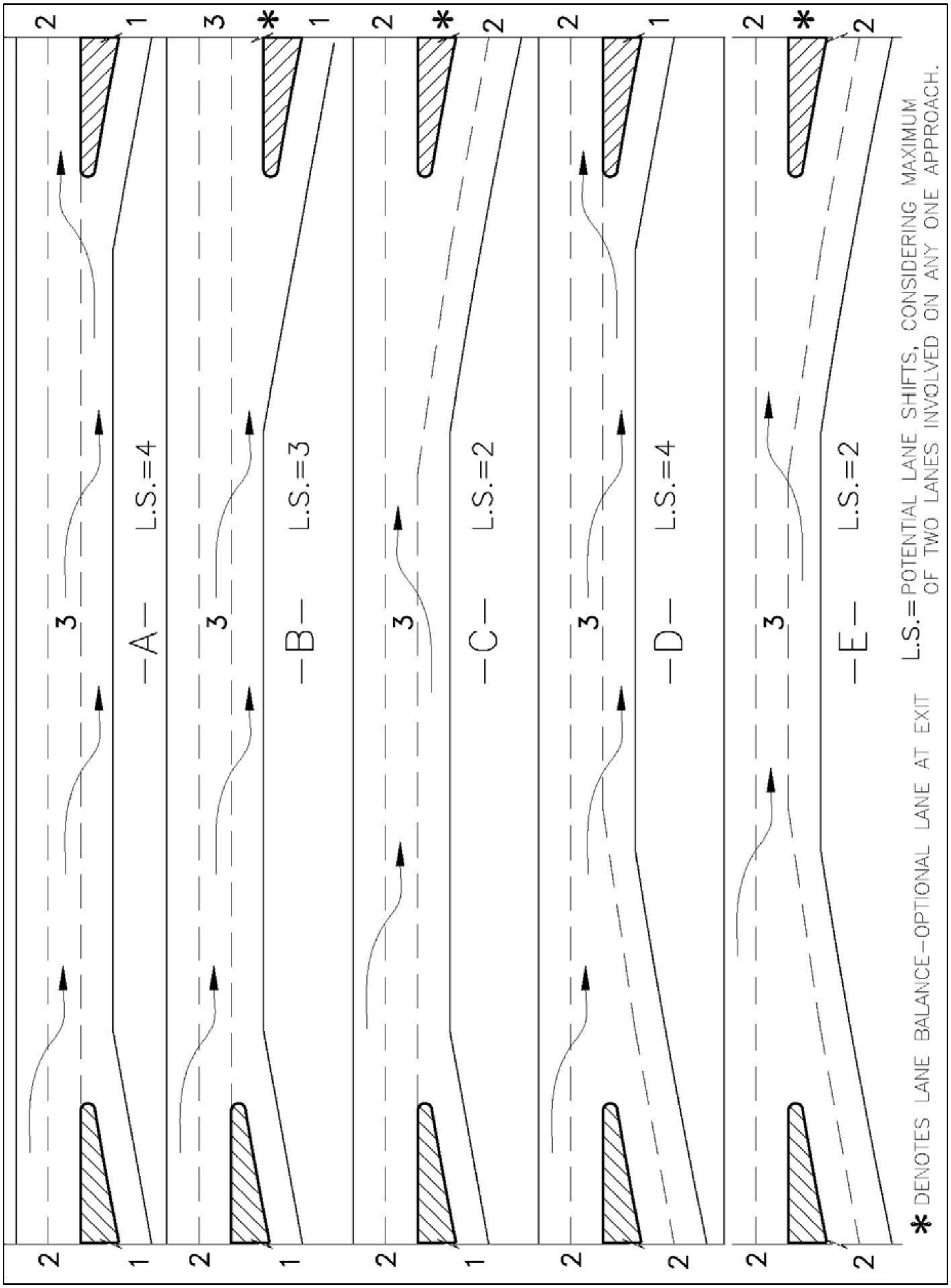


Figure 10-5A (*CALTRANS* 504. 7B (4)) Lane Configuration of Weaving Sections

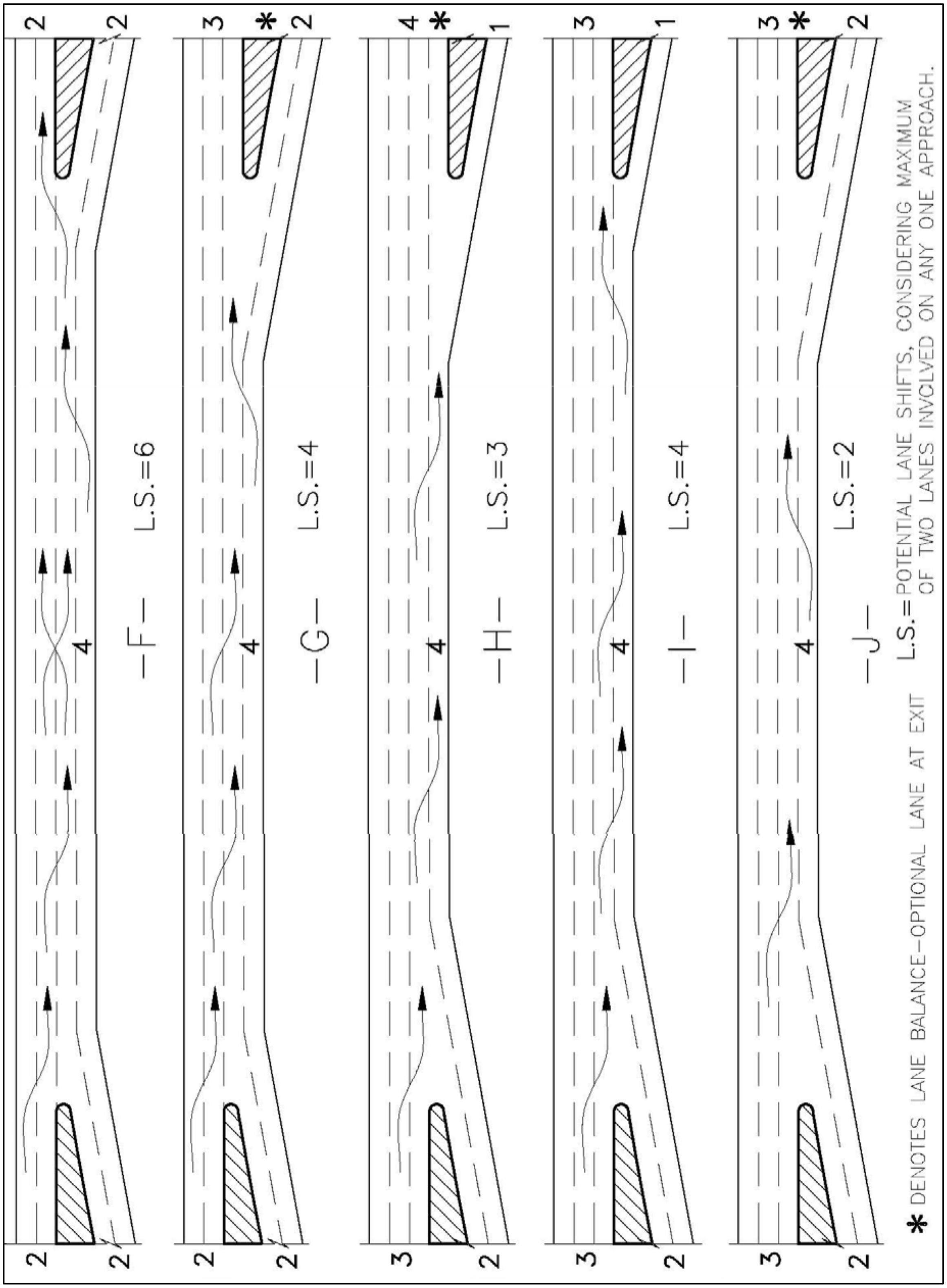


Figure 10-5B (*CALTRANS* 504. 7B (4)) Lane Configuration of Weaving Sections

10.5.10 Collector-Distributor Roads

Collector-distributor roads between two interchanges and continuous collector-distributor roads are discussed in Chapters 8 and 10 of the *PGDHS* **(1)**.

10.5.11 Two-Exit Versus Single-Exit Interchange Design

In general, interchanges that are designed with single exits are superior to those with two exits, especially if one of the exits is a loop ramp or the second exit is a loop ramp preceded by a loop entrance ramp. Whether used in conjunction with a full cloverleaf or with a partial cloverleaf interchange, the single-exit design may improve operational efficiency of the entire facility.

Additional information on this subject may be found in Chapter 10 of the *PGDHS* **(1)**.

10.5.12 Wrong-Way Entrances

Wrong-way entrance onto freeways and arterial streets is not a frequent occurrence, but it should be regarded as a serious problem whenever the likelihood exists, because each occurrence has such a high potential for culminating in a serious accident. This problem should be given special consideration at all stages of design. Most wrong-way entrances occur at freeway off ramps, at intersections at-grade along divided arterial streets, and at transitions from undivided to divided highways.

See *PGDHS* **(1)**, Chapter 10 for more information.

10.6 RAMPS

10.6.1 Types and Examples

The term "ramp" includes all types, arrangements, and sizes of turning roadways that connect two or more legs at an interchange. The components of a ramp are a terminal at each leg and a connecting road, usually with some curvature, and on a grade. Generally, the horizontal and vertical alignment standards of ramps are below that of the intersecting highway, but in some cases it may be equal. The basic types of ramps are:

- Diagonal
- One quadrant
- Loop and semidirect
- Outer connection
- Directional

For further information on these basic ramp types, see Chapter 10 of the *PGDHS* **(1)**.

10.6.2 General Ramp Design Considerations

10.6.2.1 Design Speed

Desirably, ramp design speeds should approximate the low-volume running speed on the intersecting highways. This design speed is not always practical, and lower design speeds may be selected, but they should not be less than the low-range presented in Table 10-1. Only those values for highway design speeds of at least 50 mph apply to freeway and expressway exits.

Consider the following when applying the values in Table 10-1 to various conditions and ramp types.

- **Portion of ramp to which design speed is applicable.** Values in Table 10-1 apply to the sharpest, or controlling, ramp curve, usually on the ramp proper. These speeds do not pertain to the ramp terminals, which should be properly transitioned and provided with speed-change facilities adequate for the highway speed involved.
- **Ramps for right turns.** An upper range value of design speed is often attainable on ramps for right turns, and a value between the upper and lower range is usually practical. The diamond ramp of a diagonal interchange may also be used for right turns. For these diagonal ramps, a value in the middle range is usually practical.
- **Loops.** Upper-range values of design speed generally are not attainable on loop ramps. Ramp design speeds above 30 mph for loops involve large areas, rarely available in urban areas, and long loops, which are costly and require left-turning drivers to travel a considerable extra distance. Minimum values usually control, but for highway design speeds of more than 50 mph, the loop design speed preferably should be no less than 25 mph (150-ft radius). If less restrictive conditions exist, the loop design speed and the radius may be increased.
- **Semi-direct connections.** Design speeds between the middle and upper ranges shown in Table 10-1 should be used. A design speed less than 30 mph should not be used. Generally, for short single lane ramps, a design speed greater than 50 mph is not practical. For two-lane ramps, values in the middle and upper ranges are appropriate.
- **Direct connections.** Design speeds between the middle and upper ranges shown in Table 10-1 should be used. The minimum design speed preferably should be 40 mph.
- **Different design speeds on intersecting highways.** The highway with the greater design speed should be the control in selecting the design speed for the ramp as a whole. However, the ramp design speed may vary, the portion of the ramp closer to the lower speed highway being designed for the lower speed. This variation in ramp design speed is particularly applicable where the ramp is on an upgrade from the higher speed highway to the lower speed highway.
- **At-grade terminals.** Where a ramp joins a major cross-road or street, forming an intersection at grade, Table 10-1 is not applicable to that portion of the ramp near the intersection because a stop sign or signal control is normally employed. This terminal design should be predicated on near-minimum turning conditions, as given in Chapter 9. In urban areas, where the land adjacent to the interchange is developed commercially, provisions for pedestrian movements through the interchange area should be considered.

If lower range design speeds are used for ramps, consideration should be made for additional acceleration/deceleration length and warning signs.

Range	Ramp Design Speed (mph) For Particular Highway Design Speed (mph)									
	30	35	40	45	50	55	60	65	70	75
Upper Range (85%*)	25	30	35	40	45	48	50	55	60	65
Middle range (70%*)	20	25	30	33	35	40	45	45	50	55
Lower Range (50%*)	15	18	20	23	25	28	30	30	35	40
Corresponding Minimum Radius (ft)	See Table 3-7 in the *PGDHS* (**1**)									
* percentage of highway design speed										

Table 10-1 [Table 10-1 of the *PGDHS* (1)] Guide Values for Ramp Design Speed as Related to Highway Design Speed

10.6.2.2 Curvature

See the *PGDHS* **(1),** Chapter 10.

10.6.3 Stopping Sight Distance

Stopping sight distance along a ramp should be at least as great as the design stopping sight distance. Stopping sight distance consistent with the design speed as shown in Table 3-1 shall be provided on each ramp of an interchange. Sight distance for passing is not required. There should be a clear view of the entire exit terminal, including the exit nose and a section of the ramp pavement beyond the gore. Decision sight distance should be provided when drivers must make complex or instantaneous decisions within interchanges.

If the exit terminal is signalized or stop controlled, design the terminal as an at-grade intersection and refer to Chapter 9 of this guide or the *PGDHS* **(1)**.

10.6.4 Ramp Profiles

Ramp profiles generally consist of a section of tangent grade between the vertical curves. The tangent or controlling grade on ramps should be as flat as feasible, but may be steeper than on the through facilities. Adequate sight distance is more important than a specific gradient control and should be favored in design. Consider the following:

- It is desirable that upgrades on ramps with a design speed of 45 to 50 mph be limited to 3 to 5 percent.
- Upgrades on ramps having a design speed of 40 mph should be limited to 4 to 6 percent
- Upgrades on ramps having design speeds of 25 to 30 mph should be limited to 5 to 7 percent.
- Upgrades on ramps having a design speed of 15 to 25 mph should be limited to 6 to 8 percent.
- Downgrades preferably should be limited to 3 to 4 percent on ramps having sharp horizontal curvature and significant heavy truck or bus traffic. Short upgrades of as much as 5 percent do not unduly interfere with truck and bus operation.

- Ramps with high design speeds or those joining high-speed highways generally should have flatter grades than ramps with low design speeds or minor, light-volume ramps.

Usually ramp profiles assume the shape of the letter "S," with a sag vertical curve at the lower end and a crest vertical curve at the upper end. Additional vertical curves may be necessary, particularly on ramps that cross under or over other roadways. Where a crest or sag vertical curve extends onto a ramp terminal, the length of the curve should be determined by using a design speed intermediate between those on the ramp and the highway. Minimum lengths of crest vertical curves on ramps are based on stopping sight distance as shown in Chapter 3 of this Guide.

The design controls for sag vertical curves differ from those for crests; therefore, separate design values are needed. Minimum values of sag vertical curves are based on values of K and stopping sight distance as shown in Chapter 3 of this Guide.

Profiles of ramp terminals should be designed in association with horizontal curves to avoid sight restrictions that will adversely affect operations. At an exit onto a ramp on a descending grade, a horizontal curve ahead should not appear suddenly to a driver using the ramp. Instead, the initial crest vertical curve should be made longer and sight distance over it increased so that the beginning and the direction of the horizontal curve are obvious to the driver in time for safe operations. At an entrance terminal from a ramp on an ascending grade, the portion of the ramp and its terminal intended for acceleration should closely parallel the through lane profile to permit entering drivers to have a clear view ahead, to the side, and to the rear on the through road.

A “platform” area should be provided at the at-grade terminal, approach nose, and merging end of a ramp. This platform should be an area on which the profile and cross-slope do not greatly differ from that of the through traffic lane. The length of this platform should be determined from the type of traffic control and the capacity at the terminal, but is typically at least 200 feet. For further discussion, see Chapter 9, Figure 9-44 through 9-47, of the *PGDHS* **(1)**.

In addition, an analysis of each alternate preliminary plan should be made for aesthetics. A ramp design that meets all design requirements may have objectionable features which can be eliminated with a minimum of change. Examples of objectionable features are:

- Humps or rolls in a ramp profile
- Short reverse curvature in ramp alignment

Where the main roadway in level terrain is taken over a cross road, an undesirable hump may appear in the ramp profile unless the ramp exit splits away from the main roadway before the main roadway begins to rise.

Short reverse curvature introduced in ramp alignment to obtain separation of ramp and main roadway in a short distance should be avoided, as it is impossible to obtain proper superelevation of the curves without an intervening length of tangent for superelevation transitions between the reversing curves.

10.6.5 Superelevation and Cross Slope

Consider the following for cross slope design on ramps:

- Superelevation as related to curvature and design speed on ramps is given in CDOT Standard Plans – M & S Standards (5). Where drainage impacts to adjacent property or the frequency of slow moving vehicles are important considerations, the superelevation rates and corresponding radii in Figure 3-16 of the *PGDHS* **(1)** can be used.
- The cross slope on portions of ramps on tangent normally should be sloped one way at a practical rate ranging from 1.5 to 2 percent for high-type pavements.
- In general the rate of change in cross slope in the superelevation runoff section should be based on the maximum relative gradients listed in Table 3-15 of the *PGDHS* **(1)**. The values listed in this table are applicable to single lane rotation. The adjustment factors b_w listed in Table 3-16 of the *PGDHS* **(1)** allow for slight increases in the effective gradient for wider road widths. The effective maximum relative gradients (equal to $\Delta \div b_w$) applicable to a range of roadway widths are listed in Table 9-19 of the *PGDHS* **(1)**. The superelevation development is started or ended along the auxiliary lane of the ramp terminal. Alternate profile lines for both edges should be studied to ensure that all profiles match the control points and that no unsightly bumps and dips are inadvertently developed. Spline profiles are very useful in developing smooth lane/shoulder edges.
- Another important control in developing superelevation along the ramp terminal is that of the crossover crown line at the edge of the through traffic lane. The maximum algebraic difference in cross slope between the auxiliary lane and the adjacent through lane is shown in Table 10-2.
- Three segments of a ramp should be analyzed to determine superelevation rates that would be compatible with the design speed and the configuration of the ramp. The exit terminal, the ramp proper, and the entrance terminal should be studied in combination to ascertain the appropriate design speed and superelevation rates.
- Drainage and icing issues should be considered when transitioning between superelevations.

Design Speed of Exit or Entrance Curve (mph)	Maximum Algebraic Difference in Cross Slope at Crossover Line (%)
20 and under	5.0 to 8.0
25 and 30	5.0 to 6.0
35 and over	4.0 to 5.0

Table 10-2 [Table 9-20 of the *PGDHS* (1)] Maximum Algebraic Difference in Pavement Cross Slope at Turning Roadway Terminals

10.7 INTERCHANGE DESIGN CRITERIA

10.7.1 General

The following design criteria pertain to design elements of all interchanges. Geometric and structure criteria for the design of the through highway within the interchange area are presented elsewhere in this Guide. Ramp design criteria are developed in section 10.6.

Interchanges are composed of a combination of channelization elements. For this reason, design criteria pertaining to intersections at-grade (see Chapter 9) will not be repeated here; only additional design criteria unique to interchanges will be given.

10.7.2 Sight Distance

Sight distance on the highways through a grade separation or interchange should be at least as long as that required for stopping and preferably longer. Where exits are involved, decision sight distance is preferred, although not always practical. Design of the vertical alignment is the same as that at any other point on the highway. Ramp terminals at crossroads should be treated as at-grade intersections and should be designed in accordance with Chapter 9 of the *PGDHS* **(1)**, with specific attention to intersection sight distance.

At underpasses, care should be taken to ensure that the vertical sight distance is not limited by the bottom of the girders of the overpassing structure. This may occur at locations where the highway is depressed for a short distance and the maximum grades and minimum sag vertical curves are used under the structure. Particular attention should be given to trucks, where the sight distance will be further limited due to the higher driver's height of eye.

The horizontal sight distance limitations of piers and abutments at curves usually present a more difficult problem than that of vertical limitations. With curvature of the maximum degree for a given design speed, the normal lateral offset of piers or abutments of underpasses does not provide the minimum stopping sight distance.

Similarly, on overpasses with the sharpest curvature for the design speed, sight deficiencies result from the usual practical shoulder offset to the bridge rails. This factor emphasizes the need for use of below-maximum curvature on highways through interchanges. If sufficiently flat curvature cannot be used, the clearances to abutments, piers, or bridge rails should be increased as necessary to obtain the proper sight distance even though it is necessary to increase span lengths or structure widths.

Normally, no more than 12 feet will be allowed on overpass structures for the lateral offset from the lane line to the bridge rail. Exceptions will be made for future lanes or for construction phasing requirements when replacing existing structures. See subsection 10.3 on lateral clearances on structures for additional information on this subject.

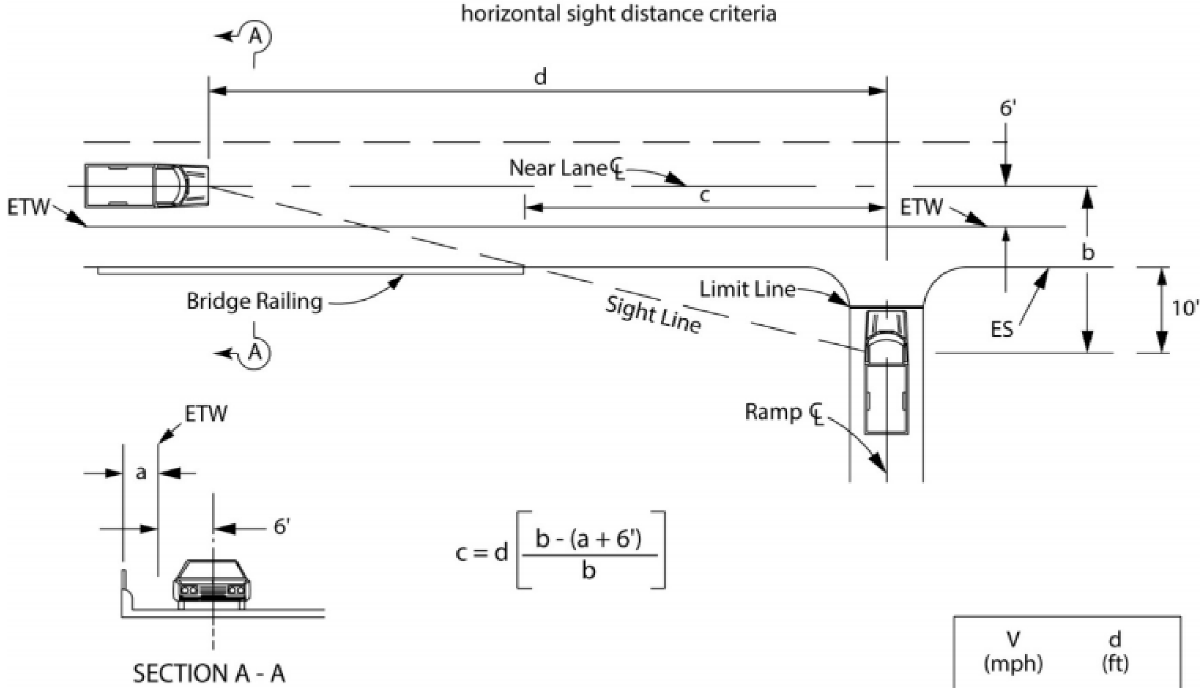
10.7.3 Sight Distance to Exit Nose

There should be a clear view of the entire exit ramp terminal, including the exit nose and a section of the ramp pavement beyond the gore. The sight distance on a freeway preceding the approach nose of an exit ramp should exceed the minimum for the through traffic design speed, desirably by 25 percent or more. In addition, the minimum sight triangle as shown in Figures 9- 5A and 5B, 10-6 and 10-7 should be provided between vehicles approaching the ramp intersections. For considerations of longer sight distances, refer to Chapter 3.

Decision sight distance given in Chapter 3 is preferred at all freeway exits and branch connections. In all cases, sight distance is measured to the center of the ramp lane right of the nose. See Figures 9-5A and 9-5B in this Guide.

Location of Ramp Intersections on the Crossroads

Unsignalized and based on 7.5 second horizontal sight distance criteria



$$c = d\left[\frac{b - (a + 6')}{b}\right]$$

a = Distance from edge of traveled way to bridge railing.

b = Distance from center of near lane to eye of ramp vehicle driver. Ramp driver's eye is assumed to be located 10′ from the edge of shoulder, but not less than 15′ from the ETW (therefore, b = 6′ + shoulder width + 10′). See Index 405.1.

c = Ramp set back from end of bridge railling.

d = Corner Sight distance along highway from intersection. (See Table above) Sight distance is measured from a 3½′ eye height on the ramp to a 4¼′ object height on the crossroad.

V = Anticipated prevailing speed on crossroad.

V (mph)	d (ft)
25	275
30	330
35	385
40	440
45	495
50	550
55	605
60	660
65	715
70	770

Figure 10-6 (*CALTRANS* 504.3I (4)) Location of Ramp Intersections on the Cross Road

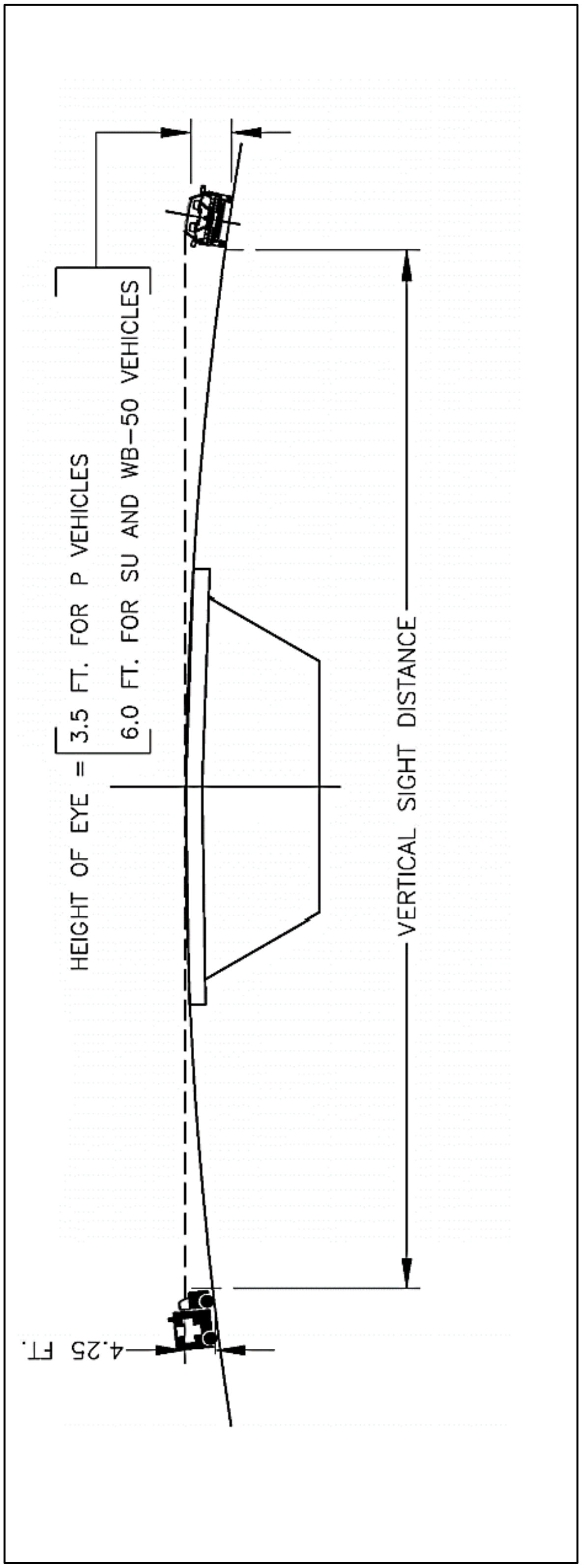


Figure 10-7 Measurement of Sight Distance at Ramp Terminals with Stop Controls

10.7.4 Gores

The term "gore" indicates an area downstream from the shoulder intersection points as illustrated in Figure 10-8.

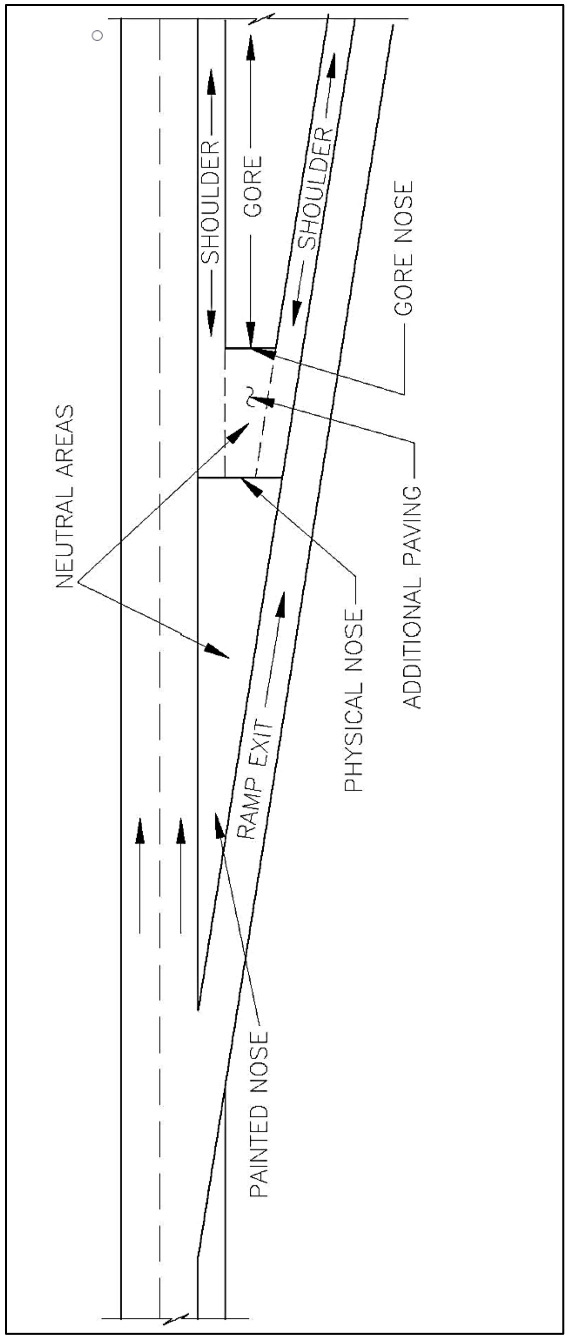


Figure 10-8 [Figure 10-61 of the *PGDHS* (1)] Typical Gore Area Characteristic

(This is a conceptual figure. See Figures 10-16 A and B for more specific details.)

It is the decision point area that must be clearly seen and understood by approaching drivers. Further, the separating ramp roadway not only must be clearly evident but also must be of geometric shape to fit the likely speeds at that point. In a series of interchanges along a freeway the gores should be uniform and have the same appearance to drivers.

As a general rule, the width at the gore nose is typically between 20 and 30 feet, including paved shoulders, measured between the traveled way of the main line and that of the ramp. This dimension may be increased if the ramp roadway curves away from the freeway immediately beyond the gore nose or if speeds in excess of 60 mph are expected to be common. See Figures 10-12 to 10-15 for additional details of gore dimensions.

The entire triangular area should be striped to delineate the proper paths on each side and to assist the driver in identifying the gore area.

It is imperative that gore areas and the areas beyond provide clear recovery area for out-of- control vehicles or for drivers who decide at the last second not to exit. Additional paving shall be placed in the neutral area between the physical nose and the gore nose to allow drivers to recover after starting their exit maneuver (see Figures 10-14 and 10-15).

The gore area and the unpaved area beyond should be kept as free of obstructions as possible to provide a clear recovery area. The unpaved area beyond the nose should be graded as nearly level with the roadways as is practicable so that vehicles inadvertently entering will not be overturned or abruptly stopped by steep slopes. Heavy sign supports, street light standards, and roadway structure supports should be kept well out of the graded gore area. Yielding or breakaway type supports should be used for the standard exit sign, and concrete footings, where used, should be kept flush with adjoining ground level. If non-yielding obstructions are unavoidable in the gore area, impact attenuators should be considered.

Although the term "gore" generally refers to the area between a through roadway and an exit ramp, the term sometimes is also used to refer to the similar area between a through roadway and a converging entrance ramp. At an entrance terminal, the point of convergence (beginning of all paved area) is defined as the "merging end." In shape, layout, and extent, the triangular maneuver area at an entrance terminal is much like that at an exit. However, it points downstream and separates traffic streams already in lanes, thereby being less of a decision area. The width at the base of the paved triangular area is narrower, usually being limited to the sum of the shoulder widths on the ramp and freeway plus a narrow physical nose 4 to 8 feet wide.

Figure 10-9 diagrammatically details a typical gore design for an entrance ramp.

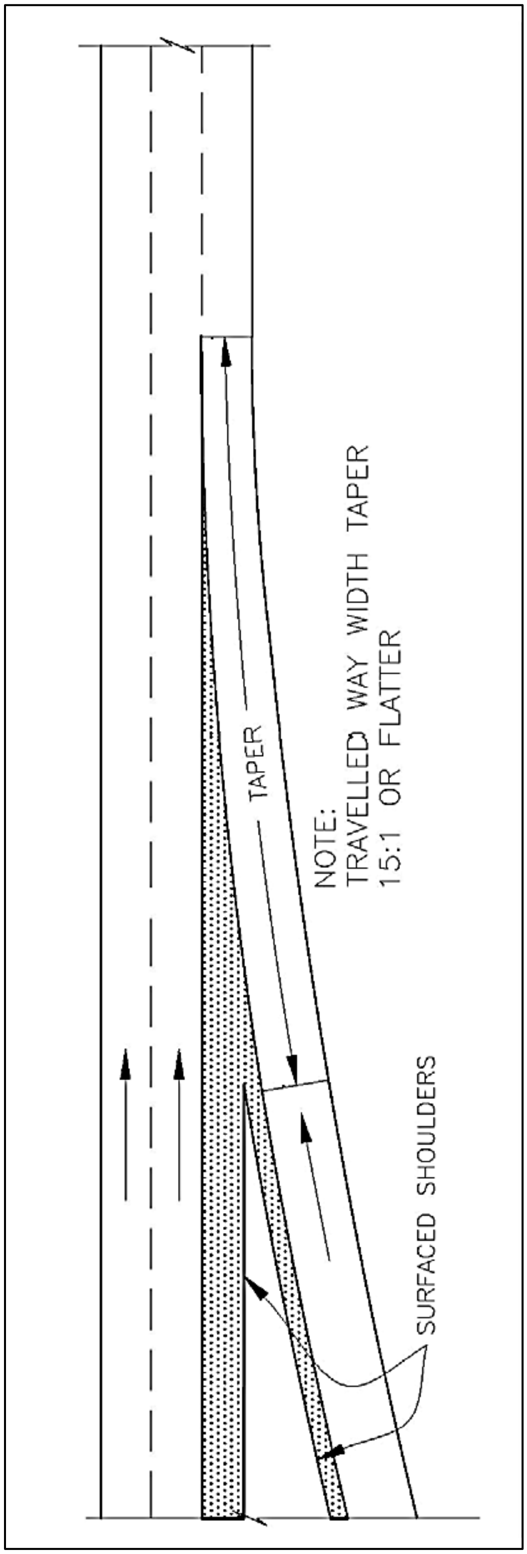


Figure 10-9 [Figure 10-63 of the *PGDHS* (1)] Traveled Way Narrowing on Entrance Ramps

10.7.5 Ramp Pavement Widths

10.7.5.1 Width and Cross Section

Ramp pavement widths are governed by the type of operation, curvature, volume, and type of traffic. It should be noted that the roadway width for a turning roadway, as distinct from pavement width, includes shoulders or equivalent clearance outside the edges of pavement. See Chapter 3 for additional information on the treatments at the edge of pavement. Design widths of ramp pavements for three general design traffic conditions are given in Table 9-1 "Design Widths of Pavements for Turning Roadways", of this guide.

10.7.5.2 Shoulders and Lateral Clearances

Design values for shoulders and lateral clearances on the ramps are as follows:

- When paved shoulders are provided on ramps, they should have a uniform width for the full length of ramp. For one-way operation, the sum of the right and left shoulder widths should not exceed 10 to 12 feet. A paved shoulder width of 2 to 4 feet is desirable on the left with the remaining width of 8 to 10 feet used for the paved right shoulder.
- The ramp traveled way widths from Table 3-29 of the *PGDHS* **(1)** for Case II and Case III should be modified when paved shoulders are provided on the ramp. The ramp traveled- way width for Case II should be reduced by the total width of both right and left shoulders. However, in no case should the ramp traveled way be less than needed for Case I. For example, with condition C and a 400-foot radius, the Case II ramp traveled-way width without shoulders is 22 feet. If a 2-foot left shoulder and an 8-foot right shoulder are provided, the minimum ramp traveled-way width should be 16 feet.
- Directional ramps with a design speed over 40 mph should have a paved right shoulder width of 8 to 10 feet and a paved left shoulder width of 1 to 6 feet.
- For freeway ramp terminals where the ramp shoulder is narrower than the freeway shoulder, the paved shoulder width of the through lane should be carried into the exit terminal. It should also begin with the entrance terminal, with the transition to the narrower ramp shoulder accomplished gracefully on the ramp end of the terminal. Abrupt changes should be avoided.
- Ramps should have a lateral clearance on the right outside of the edge of the traveled way at least 6 feet, and preferably, 8 to 10 feet, and a lateral clearance on the left of at least 4 feet beyond the edge of traveled way.
- Where ramps pass under structures, the total roadway width should be carried through the structure. Desirably, structural supports should be located beyond the clear zone. As a minimum, structural supports should be at least 4 feet beyond the edge of paved shoulder. The *AASHTO Roadside Design Guide* **(6)** provides guidance on the clear zone and the use of roadside barriers.
- Ramps on overpasses should have the full approach roadway width carried over the structure.
- Edge lines or some type of color or texture differentiation between the traveled way and shoulder is desirable.

10.7.6 Ramp Terminals

The terminal of the ramp is that portion adjacent to the through traveled way, including speed-change lanes, tapers, and islands. Ramp terminals may be the at-grade type as at the crossroad terminal of diamond or partial cloverleaf interchanges, or the free-flow type where ramp traffic merges with or diverges from high-speed through traffic at flat angles. Design elements for the former type are discussed in Chapter 9, and those for the latter type are discussed in the following sections.

Terminals are further classified according to the number of lanes on the ramp terminal, either single or multilane, and according to the configuration of the speed-change lane, either taper type or parallel type.

10.7.6.1 Right-Hand Entrances and Exits

All freeway entrances and exits shall connect to the right of through traffic. Right-hand entrances and exits operate fairly well and do not violate the concept of driver expectancy. Left-hand entrances and exits may be considered in only unusual circumstances and the design engineer should use extreme care in selecting and designing any left-hand entrances and exits.

10.7.6.2 Left-Hand Entrances and Exits

Left-hand entrances and exits are contrary to the concept of driver expectancy when intermixed with right-hand entrances and exits.

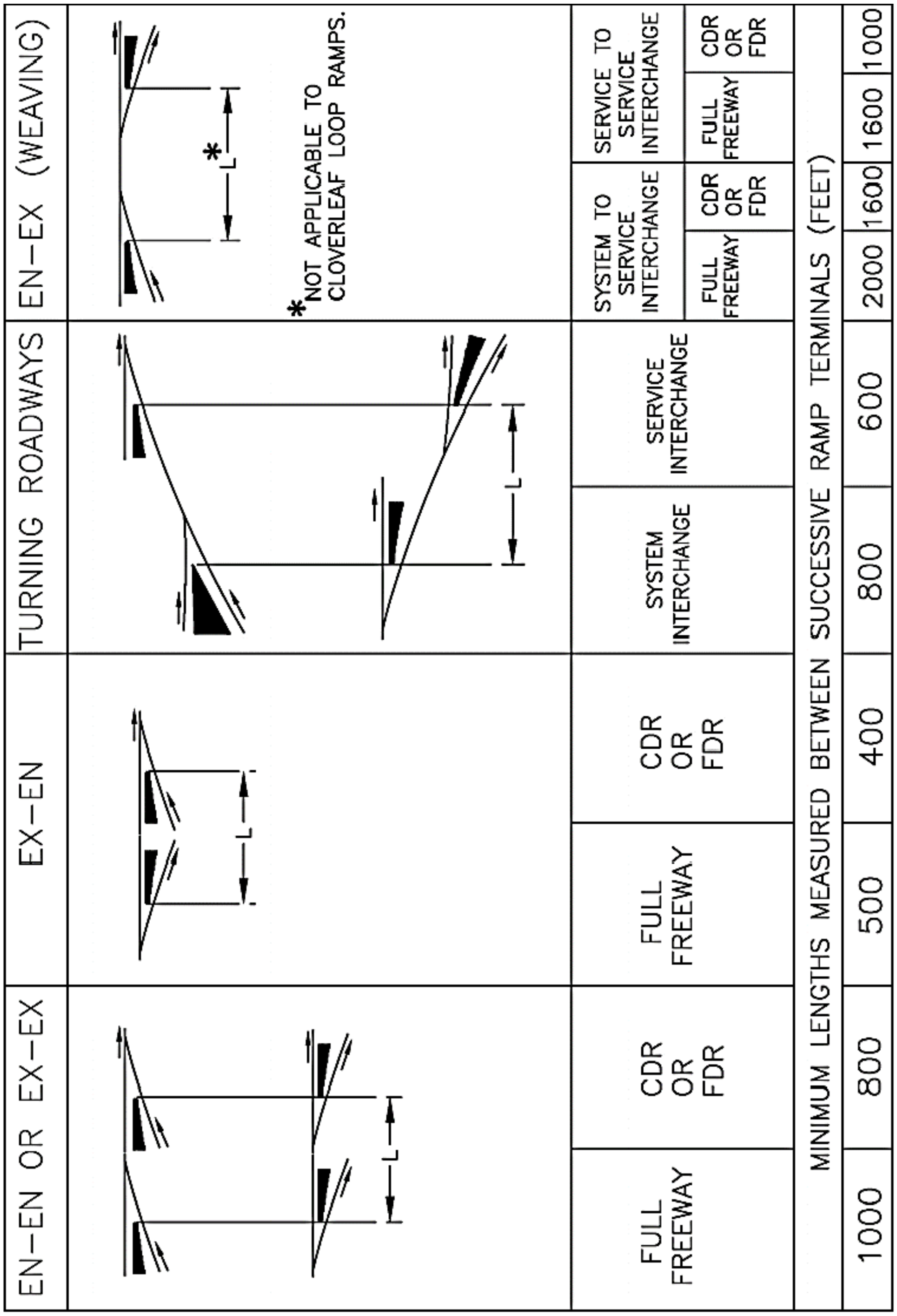
Extreme care should be exercised to avoid left-hand entrances and exits in the design of interchanges. Even in the case of major forks and branch connections, the less significant roadway should exit and enter on the right. See the discussion on route continuity in Chapter 10 of the *PGDHS* **(1)**.

Left-side terminals break up the uniformity of interchange patterns and in general create hesitant operation on the through roadways.

Left-hand entrances and exits are considered satisfactory for collector-distributor roads; however, their use on high-speed, free-flow ramp terminals is not recommended. Because left- hand entrances and exits are contrary to driver expectancy, special attention should be given to weaving from adjacent right-hand entrances, signing, and the provision for decision sight distance in order to alert the driver that an unusual situation exists.

10.7.6.3 Terminal Location and Sight Distance

Freeway entrances and exits should be located on tangent sections wherever possible to provide maximum sight distance and optimum traffic operations. Entrances and exits at left-hand curves, particularly curves requiring superelevation, should be avoided whenever possible. Ramp terminal spacing shall conform to Figure 10-10 wherever practical.



Abbreviations	
EN = Entrance	CDR = Collector- Distributor Road
EX = Exit	FDR = Freeway Distributor Road

Figure 10-10 [Figure 10-68 of the *PGDHS* (1)] Recommended Minimum Ramp Terminal Spacing

10.7.6.4 Speed-Change Lanes

Two general forms of speed change lanes are: (1) the taper and (2) the parallel type. The taper type provides a direct entry or exit at a flat angle; whereas the parallel type has an added lane for changing speed. When conditions allow, CDOT prefers the parallel type.

Speed-change lanes are provided at all ramp connections. See Tables 10-3 and 10-4 for minimum lengths. Table 10-5 gives corrections to be applied when the speed-change lane grades are 3 percent or steeper.

Highway Design Speed, mph (V)	Speed Reached, mph (V_A)	L = Deceleration Length, (ft) for Design Speed of Exit Curve, mph (V')								
		Stop Condition	15	20	25	30	35	40	45	50
		For Average Running Speed on Exit Curve, mph (V'_A)								
		0	14	18	22	26	30	36	40	44
30	28	235	200	170	140	---	---	---	---	---
35	32	280	250	210	185	150	---	---	---	---
40	36	320	295	265	235	185	155	---	---	---
45	40	385	350	325	295	250	220	---	---	---
50	44	435	405	385	355	315	285	225	175	---
55	48	480	455	440	410	380	350	285	235	---
60	52	530	500	480	460	430	405	350	300	240
65	55	570	540	520	500	470	440	390	340	280
70	58	615	590	570	550	520	490	440	390	340
75	61	660	635	620	600	575	535	490	440	390

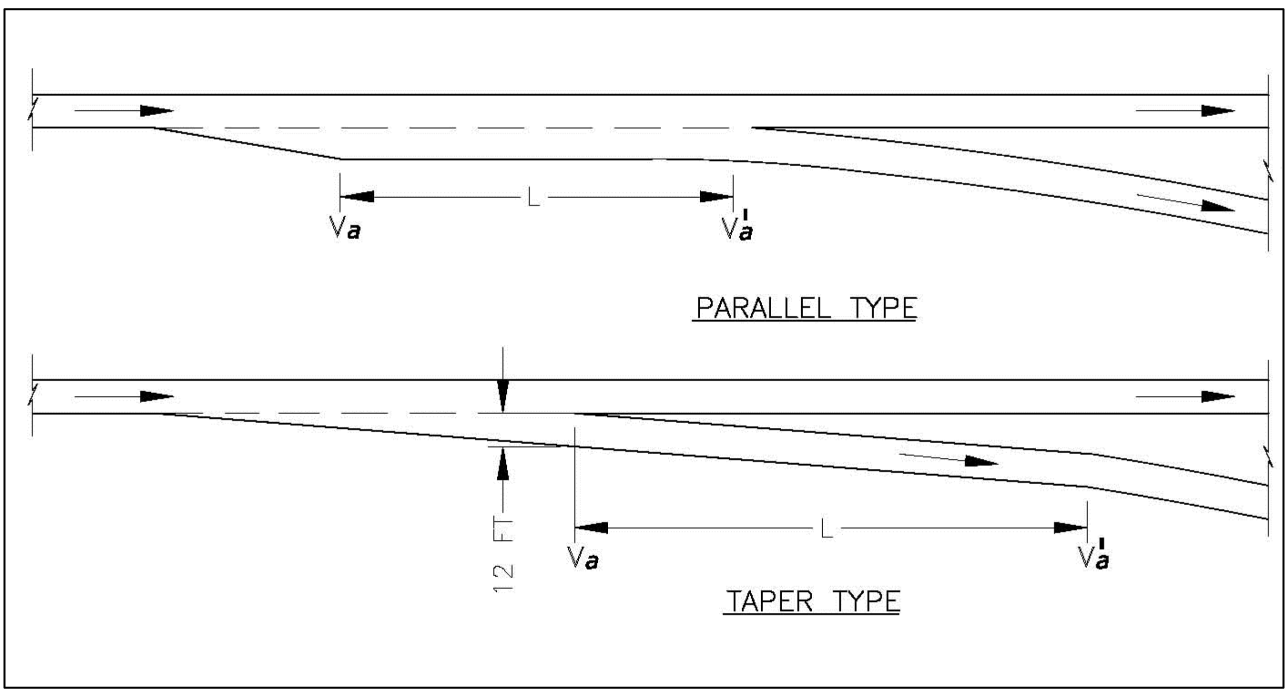


Table 10-3 [Table 10-5 of the *PGDHS* (1)] Minimum Deceleration Lengths for Exit Terminals With Flat Grades of 2 Percent or Less

Highway Design Speed, mph (V)	**Speed Reached, mph (V_A)**	**L = Acceleration Length, (ft) for Design Speed of Entrance Curve, mph**								
		Stop Condition	15	20	25	30	35	40	45	50
		and initial speed, mph (V'_A)								
		0	14	18	22	26	30	36	40	44
30	23	180	140	---	---	---	---	---	---	---
35	27	280	220	160	---	---	---	---	---	---
40	31	360	300	270	210	120	---	---	---	---
45	35	560	490	440	380	280	160	---	---	---
50	39	720	660	610	550	450	350	130	---	---
55	43	960	900	810	780	670	550	320	150	---
60	47	1200	1140	1100	1020	910	800	550	420	180
65	50	1410	1350	1310	1220	1120	1000	770	600	370
70	53	1620	1560	1520	1420	1350	1230	1000	820	580
75	55	1790	1730	1630	1580	1510	1420	1160	1040	780
Uniform 50:1 to 70:1 tapers are recommended where lengths of acceleration lanes exceed 1,300 ft.										

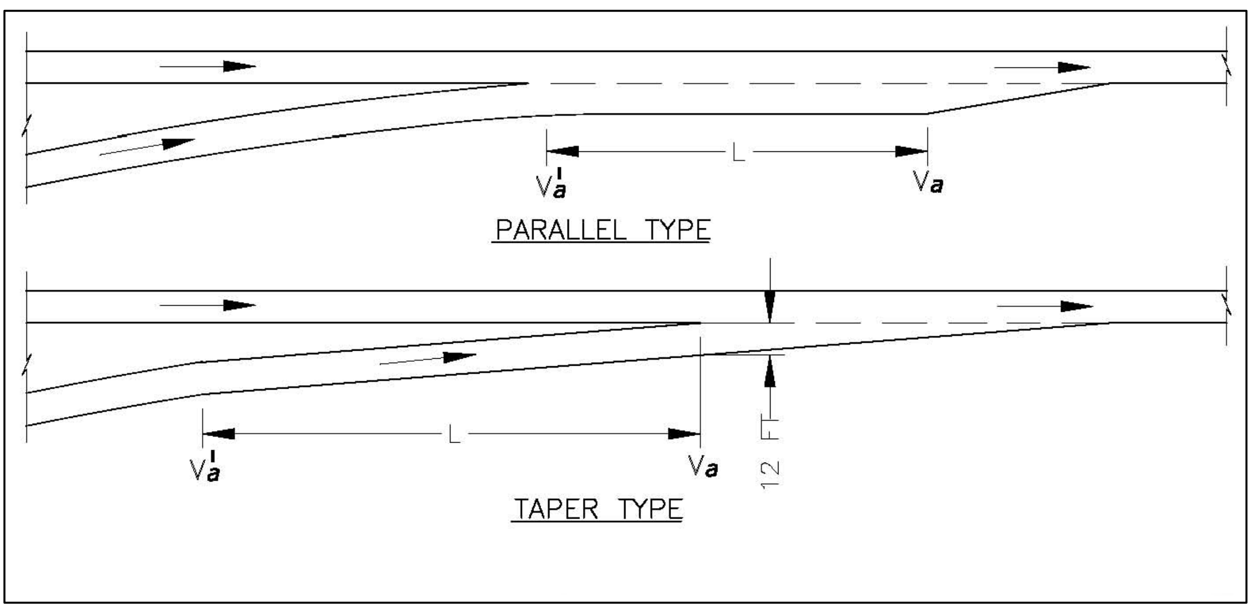


Table 10-4 [Table 10-3 of the *PGDHS* (1)] Minimum Acceleration Lengths for Entrance Terminals With Flat Grades of 2 Percent or Less

DESIGN SPEED OF HIGHWAY (mph)	DECELERATION LANES Ratio of Length on Grade to Length on Level for Design Speed of Turning Curve (mph)[a]				
All speeds	3 to 4% UPGRADE 0.9				3 to 4% DOWNGRADE 1.2
All speeds	5 to 6% UPGRADE 0.8				5 to 6% DOWNGRADE 1.35
DESIGN SPEED OF HIGHWAY (mph)	ACCELERATION LANES **Ratio of Length on Grade to Length of Level for Design Speed of Turning Curve (mph)***				
	DESIGN SPEED OF TURNING ROADWAY CURVE, mph				
	20	30	40	50	All speeds
	3 to 4% UPGRADE				3 to 4% DOWNGRADE
40	1.3	1.3	---	---	0.7
45	1.3	1.35	---	---	0.675
50	1.3	1.4	1.4	---	0.65
55	1.35	1.45	1.45	---	0.625
60	1.4	1.5	1.5	1.6	0.6
65	1.45	1.55	1.6	1.7	0.6
70	1.5	1.6	1.7	1.8	0.6
	5 to 6% UPGRADE				5 to 6% DOWNGRADE
40	1.5	1.5	---	---	0.6
45	1.5	1.6	---	---	0.575
50	1.5	1.7	1.9	---	0.55
55	1.6	1.8	2.05	---	0.525
60	1.7	1.9	2.2	2.5	0.5
65	1.85	2.05	2.4	2.75	0.5
70	2.0	2.2	2.6	3.0	0.5

* Ratio from this table multiplied by length in Table 10-3 or 10-4 gives length of speed-change lane on grade.

Table 10-5 [Table 10-4 of the *PGDHS* (1)] Speed-Change Lane Adjustment Factors as a Function of Grade

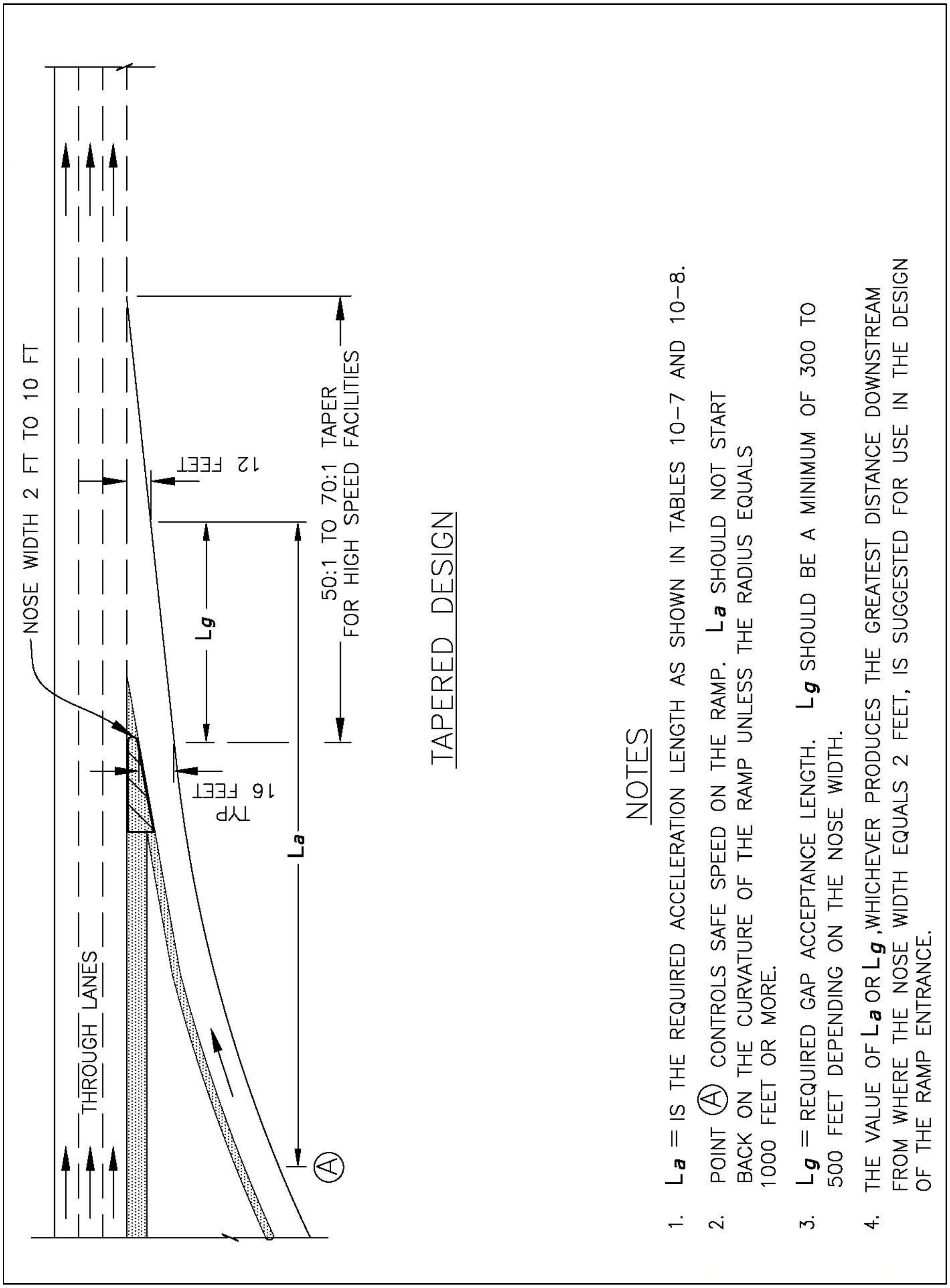


Figure 10-11A [Figure 10-69 of the *PGDHS* (1)] Typical Single-Lane Entrance Ramps (Tapered)

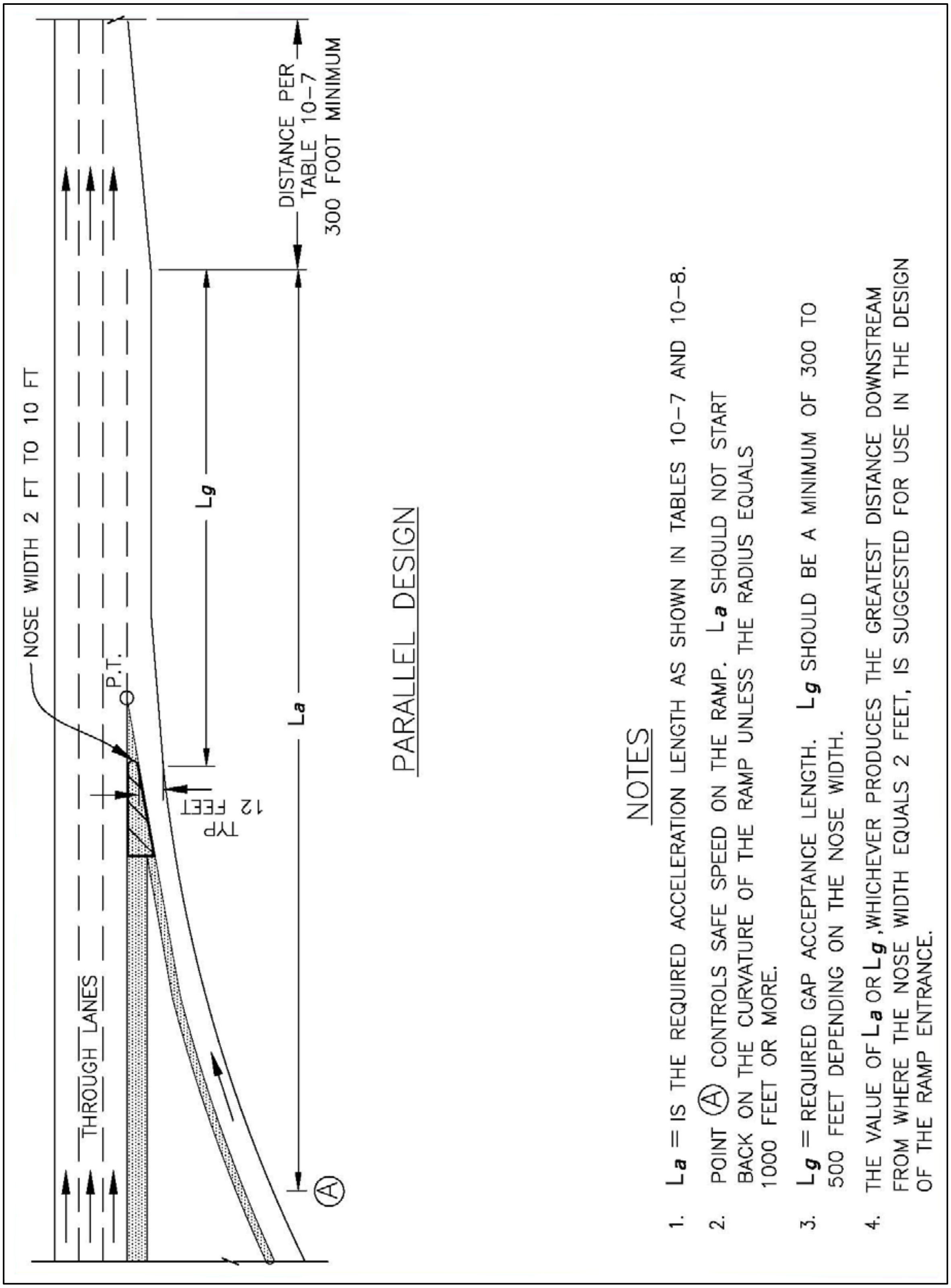


Figure 10-11B [Figure 10-69 of the *PGDHS* (1)] Typical Single-Lane Entrance Ramps (Parallel) (Preferred)

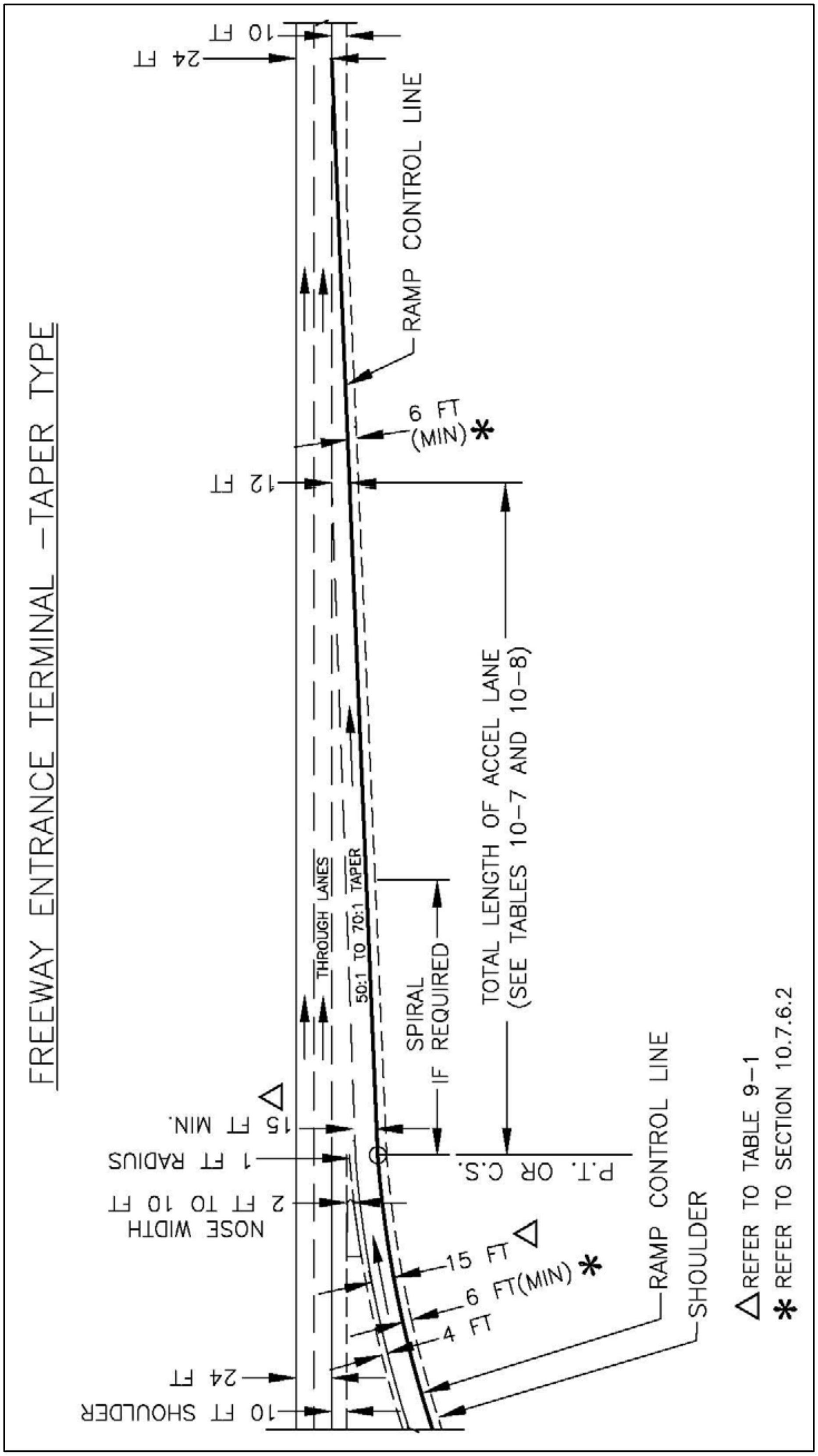


Figure 10-12 Freeway Entrance Terminal – Taper Type

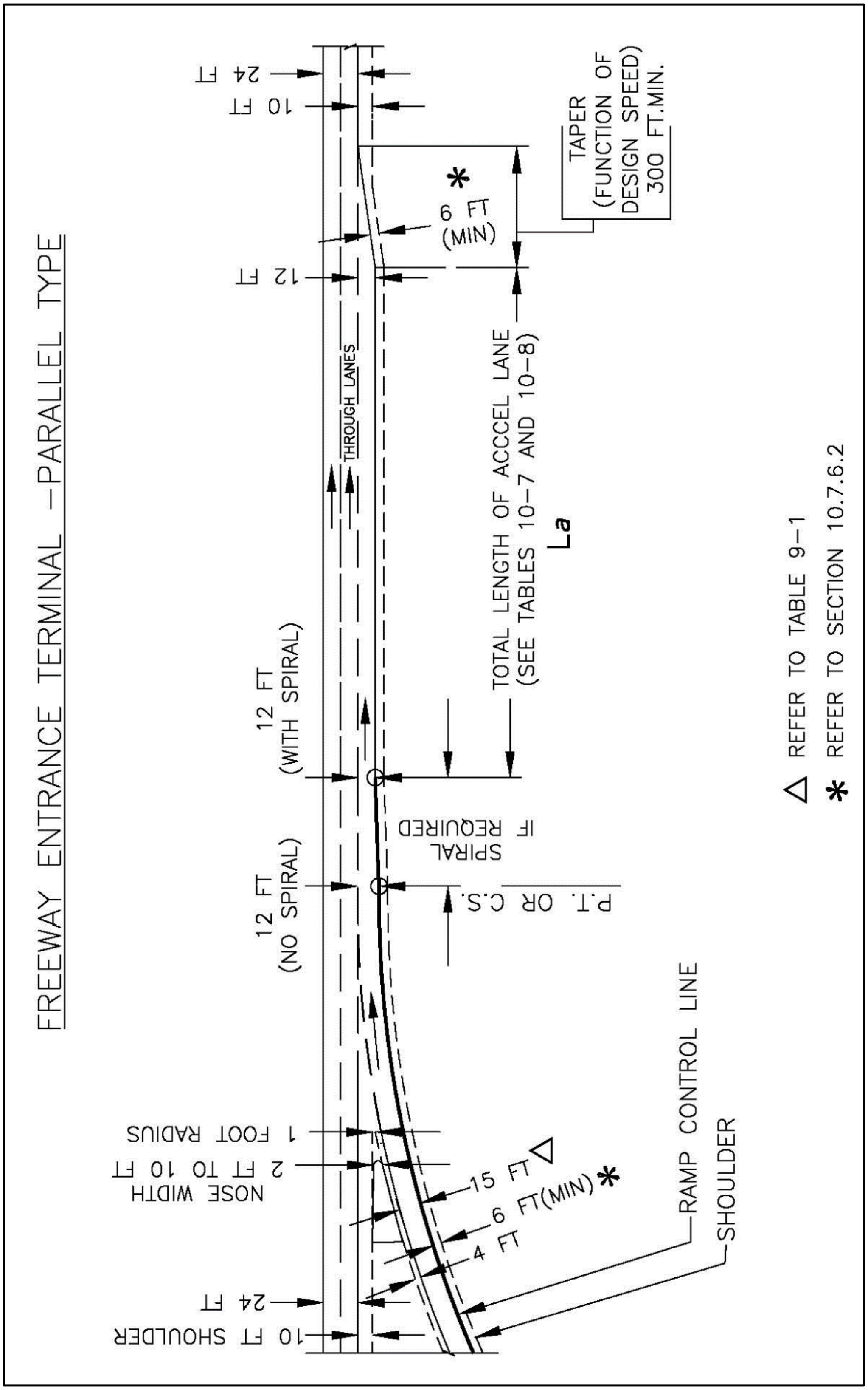


Figure 10-13 Freeway Entrance Terminal – Parallel Type (Preferred)

10.7.7 Single Lane Free-Flow Terminals, Entrances

Design of entrance ramp terminals should conform to the standard designs illustrated by Figures 10-11A, 10-11B, 10-12 and 10-13. Single lane ramps should be designed for one lane, passing permitted operation. It is up to the design engineer, with the approval of the project manager, to determine the type of ramp terminal, parallel type or taper type, at each location, although there should be an effort to obtain consistency in a corridor.

10.7.7.1 Taper Type Entrance

The taper type entrance of proper dimensions usually operates smoothly at all volumes up to and including the design capacity of merging areas. By relatively minor speed adjustment, the entering driver can see and use an available gap in the through-traffic lane. The taper type entrance allows for the ability to provide proper superelevation transitions from the curve to the tangent section in the long, triangular gore area.

10.7.7.2 Parallel Type Entrance

The parallel type provides an added lane of sufficient length to enable a vehicle to accelerate to near-freeway speed prior to merging. A taper is provided at the end of the added lane. A 300-foot taper is the normal length of taper for design speeds up to 70 mph. The parallel type entrance is preferred over the taper type entrance.

Desirably, a curve with a radius of 1000 feet or more and a length of at least 200 feet should be provided in advance of the added lane. If this curve has a short radius, motorists tend to drive directly onto the freeway without using the acceleration lane. This behavior results in undesirable merging operation. The length of the parallel lane is generally measured from the point where the left edge of the traveled way of the ramp joins the traveled way of the freeway to the beginning of the taper. If the curve approaching the acceleration lane has a long radius of 1,000 feet or more, and the motorist has an unobstructed view of traffic on the freeway to the left, a part of the ramp proper may be considered as part of the acceleration lane.

The operational and safety benefits of long acceleration lanes are well-recognized, particularly where both the freeway and ramp carry high-traffic volumes. A long acceleration lane provides more time for the merging vehicles to find an opening in the through-traffic stream. An acceleration length of at least 1,200 ft, plus the taper, is desirable whenever it is anticipated that the ramp and freeway will carry traffic volumes approximately equal to the design capacity of the merging area.

The disadvantage of the parallel type is the general inability to design and construct the entrance curve with the proper superelevation transitions. This is particularly evident when concrete pavement is used and the shoulder is paved concurrently with the mainline paving.

10.7.8 Single-Lane Free-Flow Terminals, Exits

Design of exit ramp terminals should conform to the standard designs illustrated by Figures 10-14 and 10-15. Single-lane ramps should be designed for one lane, passing permitted operation.

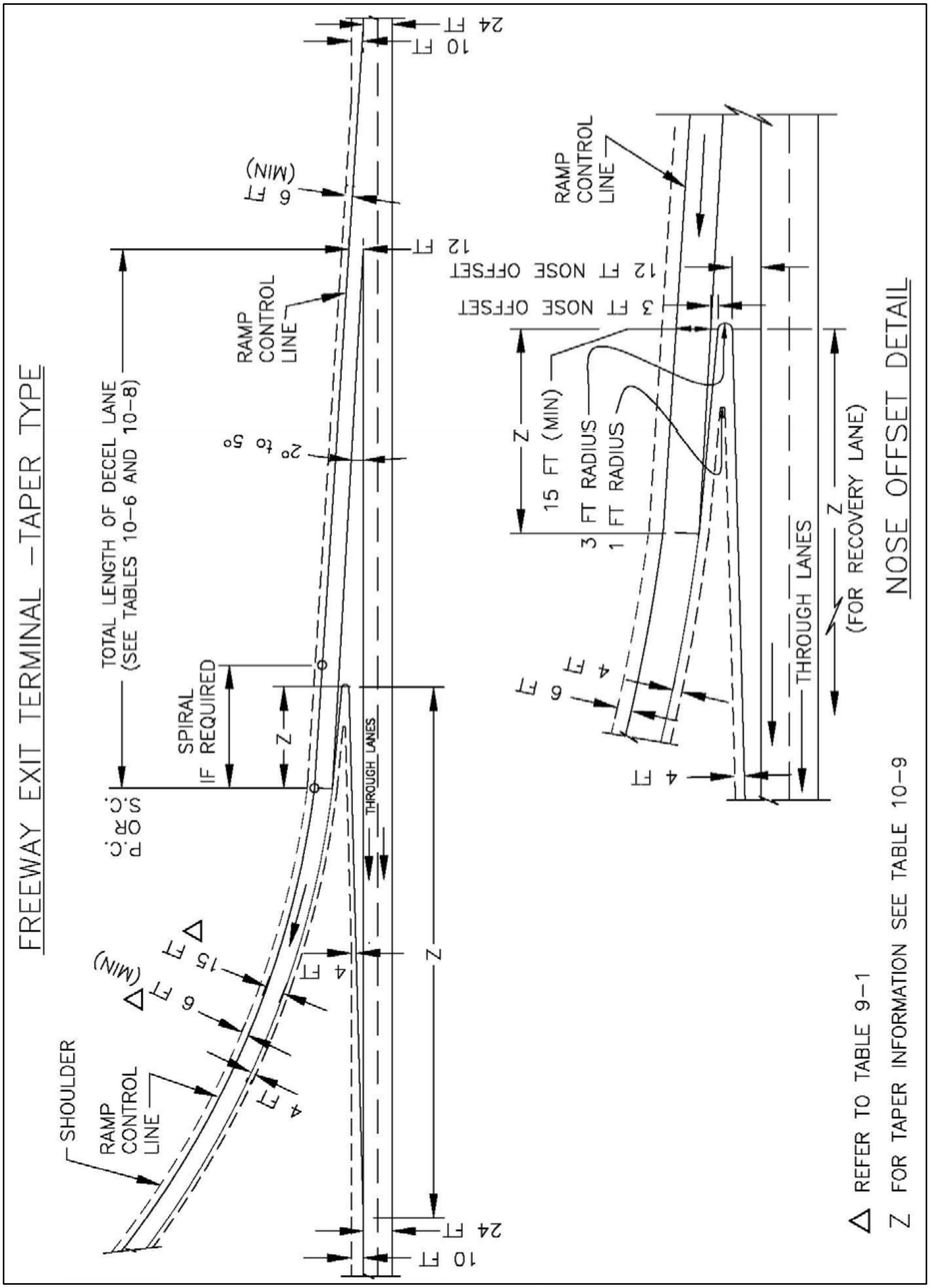


Figure 10-14 Freeway Exit Terminal – Taper Type

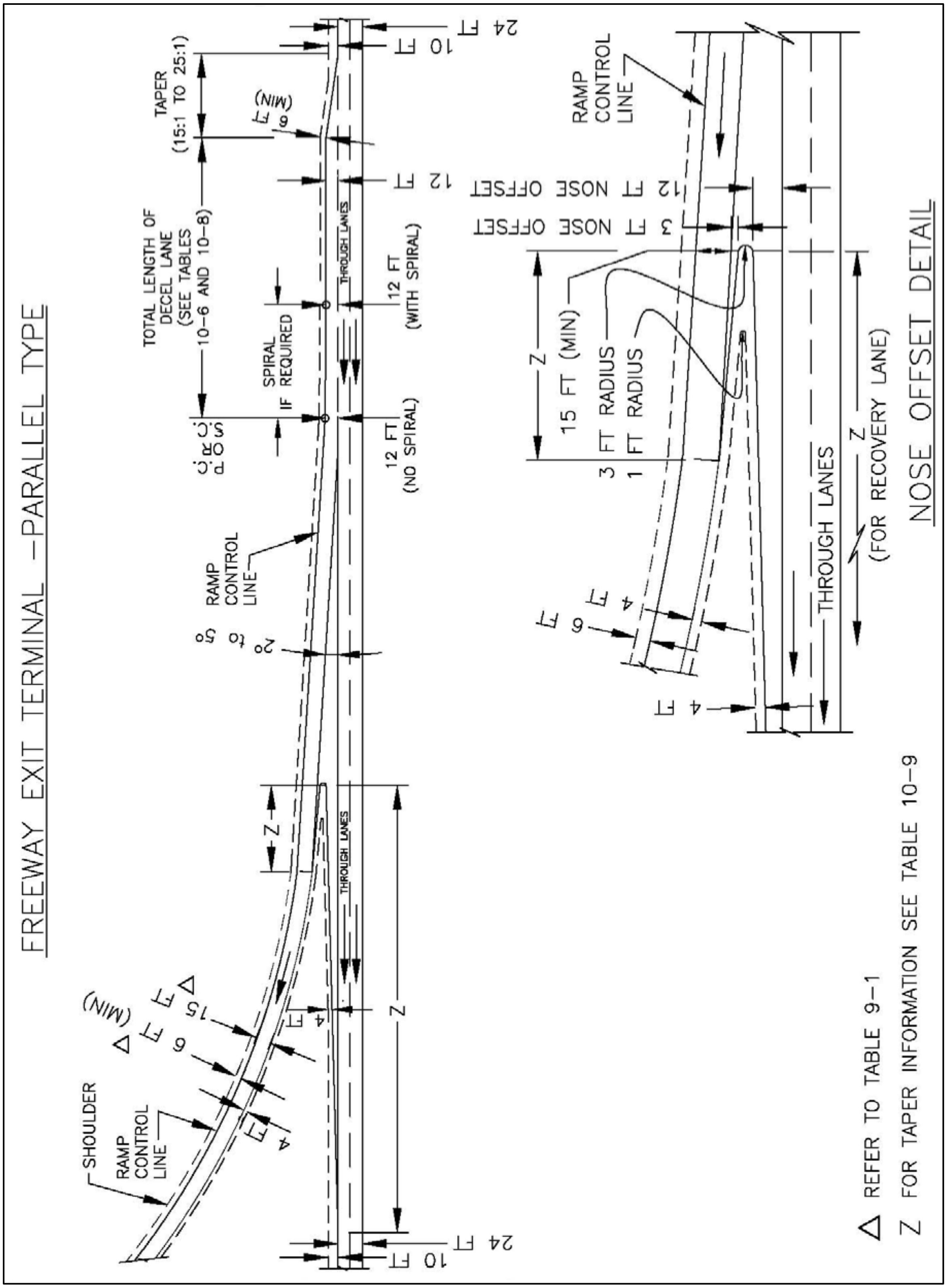


Figure 10-15 Freeway Exit Terminal – Parallel Type

Design Speed of Approach Highway	Length of Nose Taper (Z) Per Unit Width of Nose Offset
30	15.0
35	17.5
40	20.0
45	22.5
50	25.0
55	27.5
60	30.0
65	32.5
70	35.0
75	37.5

Table 10-6 [Table 10-2 of the *PGDHS* (1)] Minimum Length of Taper Beyond an Offset Nose

10.7.8.1 Taper Type Exits

The taper-type exit fits the direct path preferred by most drivers, permitting them to follow an easy path within the diverging area. Vehicles leave the through lane at a higher speed than on the parallel type thereby reducing the possibilities of rear-end collisions. Deceleration is accomplished on the taper, once the vehicle has left the through lanes. The length for deceleration begins at the point where the deceleration lane is 12 feet wide and extends to the point controlling the safe speed for the ramp, usually the PC of the exit curve. The divergence angle is usually between 2 and 5 degrees.

The taper-type exit terminal design can be used advantageously in developing the desired long, narrow, triangular emergency maneuver area just upstream from the exit nose. The taper configuration also works well in the length-width superelevation adjustments to effect a ramp cross slope different from that of the through lane.

10.7.8.2 Parallel Type Exits

A parallel-type exit terminal usually begins with a taper, followed by a derived length of added full lane that is parallel to the traveled way. This design assumes that driver will exit near the beginning of the added lane, and effect speed change thereafter. It requires a reverse-curve maneuver that is somewhat unnatural. Some drivers may choose to avoid the reverse-curve exit path and turn directly off the through lane in the vicinity of the exit nose. This may result in undesirable deceleration on the through lane, in undesirable conflict on the deceleration lane, or in excessive speed in the exit-nose area.

The length of a parallel-type deceleration lane is usually measured from the point where the added lane attains a 12-foot width to the point where the alignment of the ramp roadway departs from the alignment of the freeway. Longer parallel-type deceleration lanes are more likely to be used properly.

The taper portion of the exit should be 15:1 to 25:1.

10.7.8.3 Free-Flow Terminals on Curves

If an exit ramp is required at a sharp left-hand curve, the change in superelevation from the main line to the ramp can be troublesome. Sometimes this change in superelevation can be transitioned smoothly using a long taper-type design.

A parallel-type design in this situation usually results in adverse superelevation on the exit curve. This can result in operational problems at the exit, particularly when snow and ice are present.

If an exit ramp is required near the beginning of a curve on the mainline, a taper-type exit may cause traffic in the right-most lane to follow the ramp. In this case, a separate and parallel ramp upstream of the PC may be required. Another option would be to move the exit taper to a point in advance of the PC of the curve thus avoiding the tendency of traffic in the right-most lane to follow the ramp. See Figure 10-71 of the *PGDHS* (**1**) for layout.

10.7.8.4 At-Grade Terminals

Ramps in metropolitan areas may require additional lanes to provide storage space for vehicles waiting to cross or enter heavy city street traffic. See Figure 10-16A and 10-16B for examples of a single lane ramp exit transition to two lanes. Contact the Region Traffic Engineer for required storage lengths.

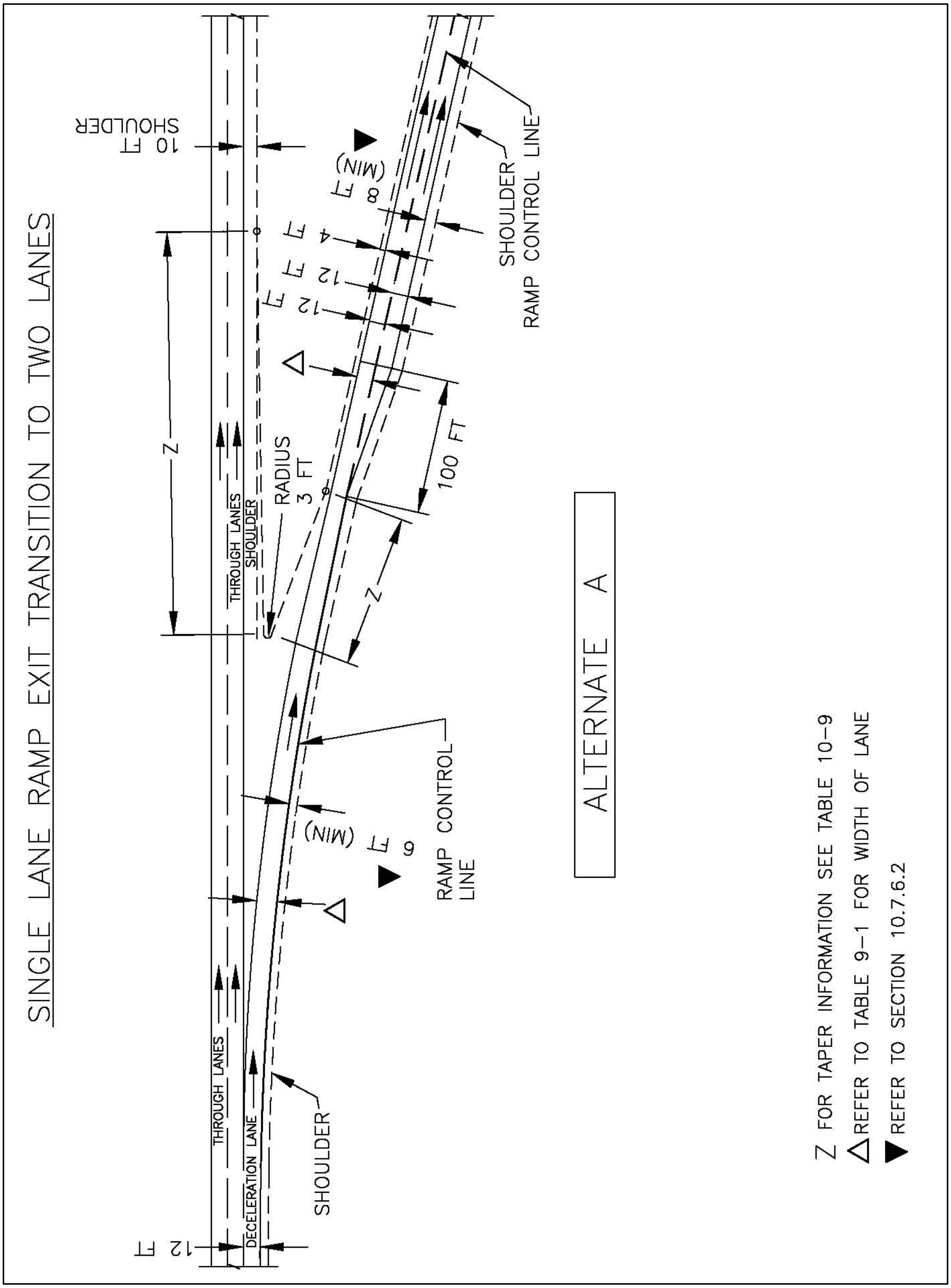


Figure 10-16A Single-Lane Ramp Exit Transition to Two Lanes (Alternate A)

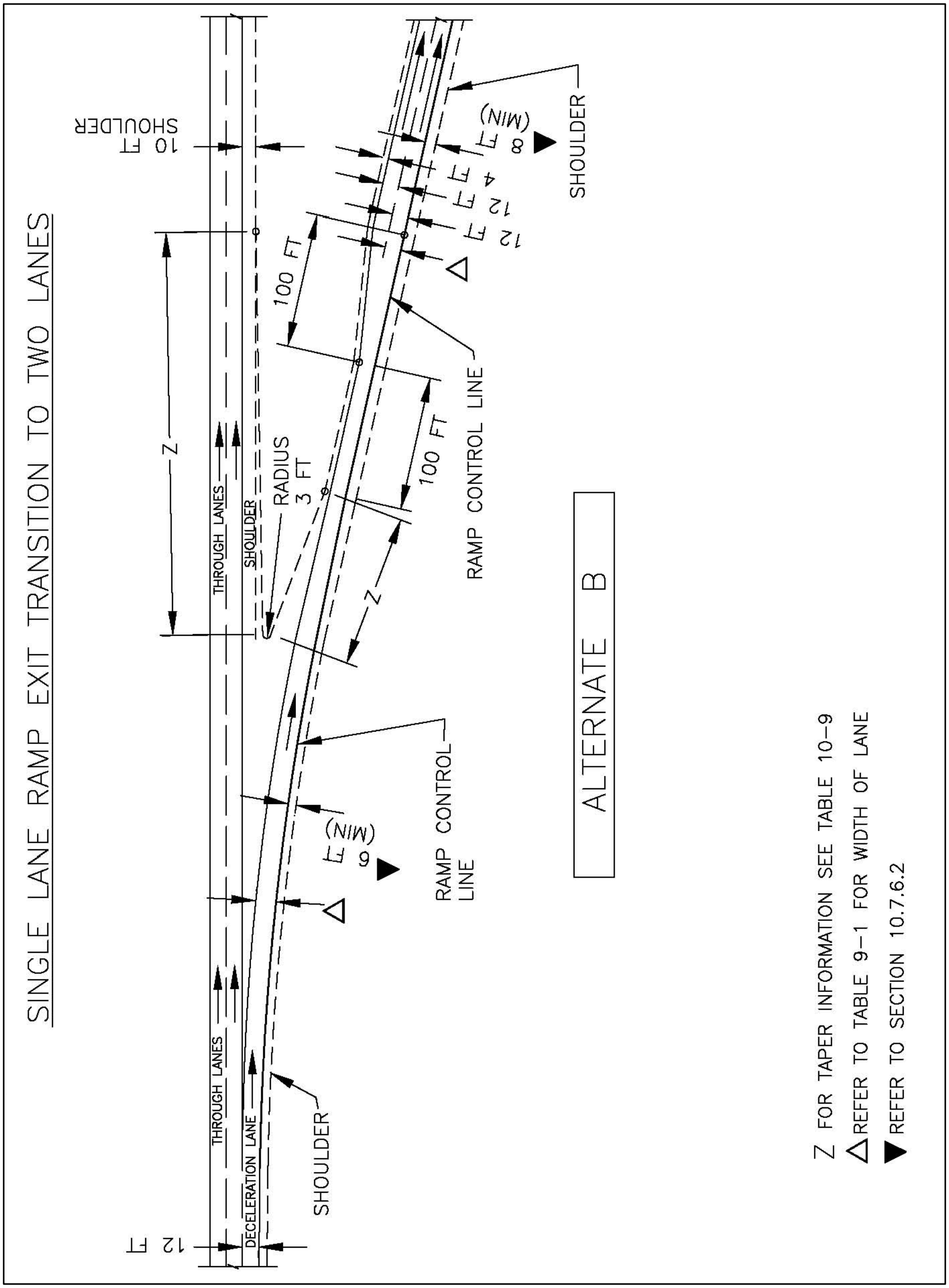


Figure 10-16B Single-Lane Ramp Exit Transition to Two Lanes (Alternate B)

10.7.9 Multilane Free-Flow Terminals

Multilane terminals are required where traffic is too great for single-lane operation. The most common multilane terminals consist of two-lane entrances and exits at freeways. Other multilane terminals are sometimes termed "major forks" and "branch connections." The latter term denotes a separation and joining of two major routes.

10.7.9.1 Two-Lane Entrances

Two-lane entrances are warranted for either a branch connection, ramp metering, or in situations created by capacity requirements on the on-ramp. To satisfy lane-balance requirements, at least one additional lane must be provided downstream. This additional lane may be a basic lane if also required for capacity, or an auxiliary lane that may be dropped 2,500 to 3,000 feet downstream or at the next interchange. In some cases, two additional lanes may be necessary because of capacity requirements. This will result in a right lane drop on the two-lane ramp, rather than a forced inside-lane merge on the classic taper-type two-lane entrance. In some cases, where volumes on the two-lane ramp are at the lower end, the outer edge of pavement may be continuously tapered, usually on a 50:1, with the striping showing a right-lane drop. In no case should a two-lane ramp be striped for an inside merge with the right lane being the continuous lane. In areas where interchanges are closely spaced, one lane may become a continuous auxiliary lane.

10.7.9.2 Two-Lane Exits

Where traffic leaving the freeway at an exit terminal exceeds the design capacity of a single lane, it is necessary to provide a two-lane exit terminal. To satisfy lane balance requirements and not reduce the basic number of through lanes, it is usually necessary to add an auxiliary lane upstream from the exit. See Figure 10-18.

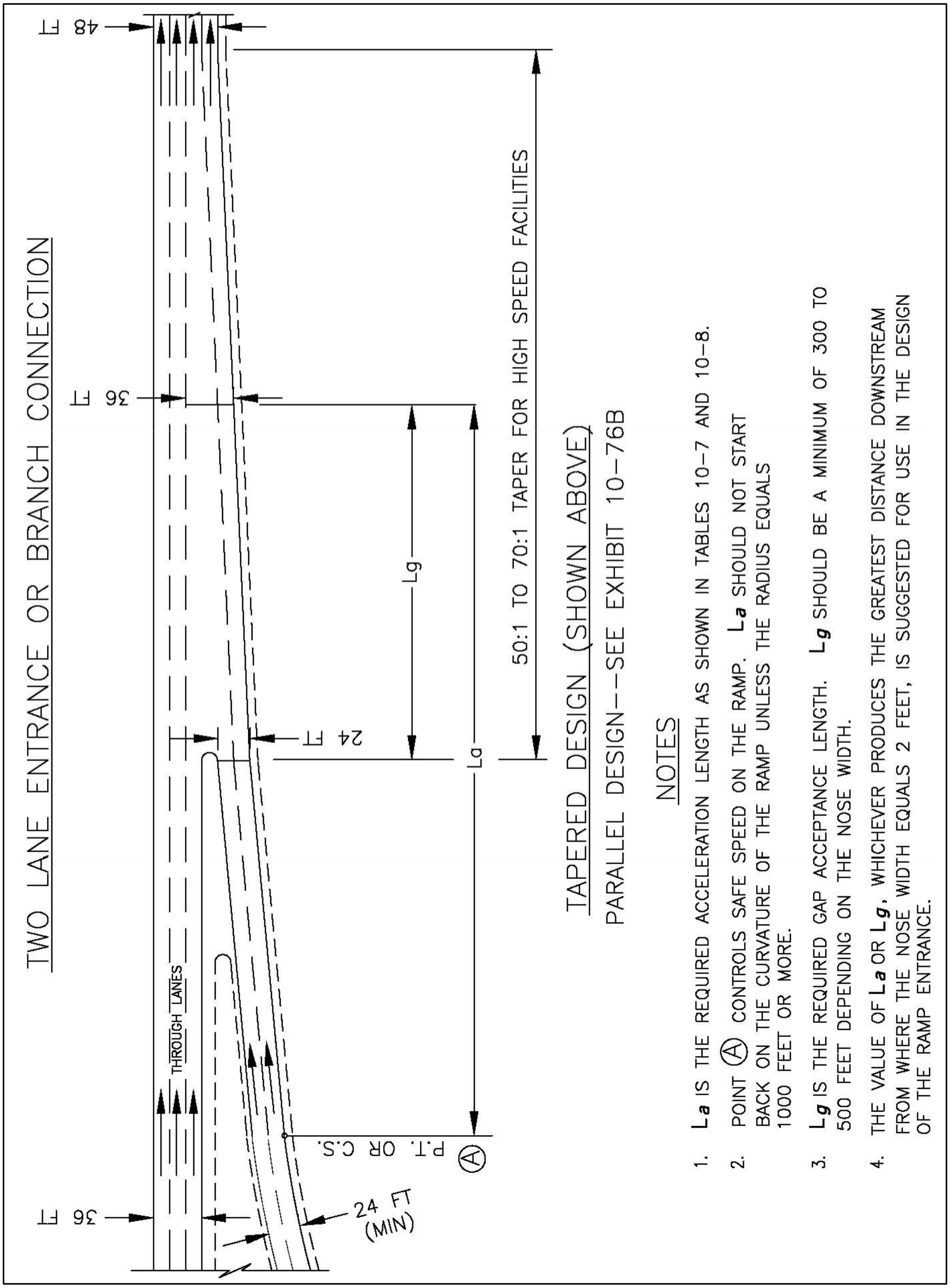


Figure 10-17 Two-Lane Entrance or Branch Connection

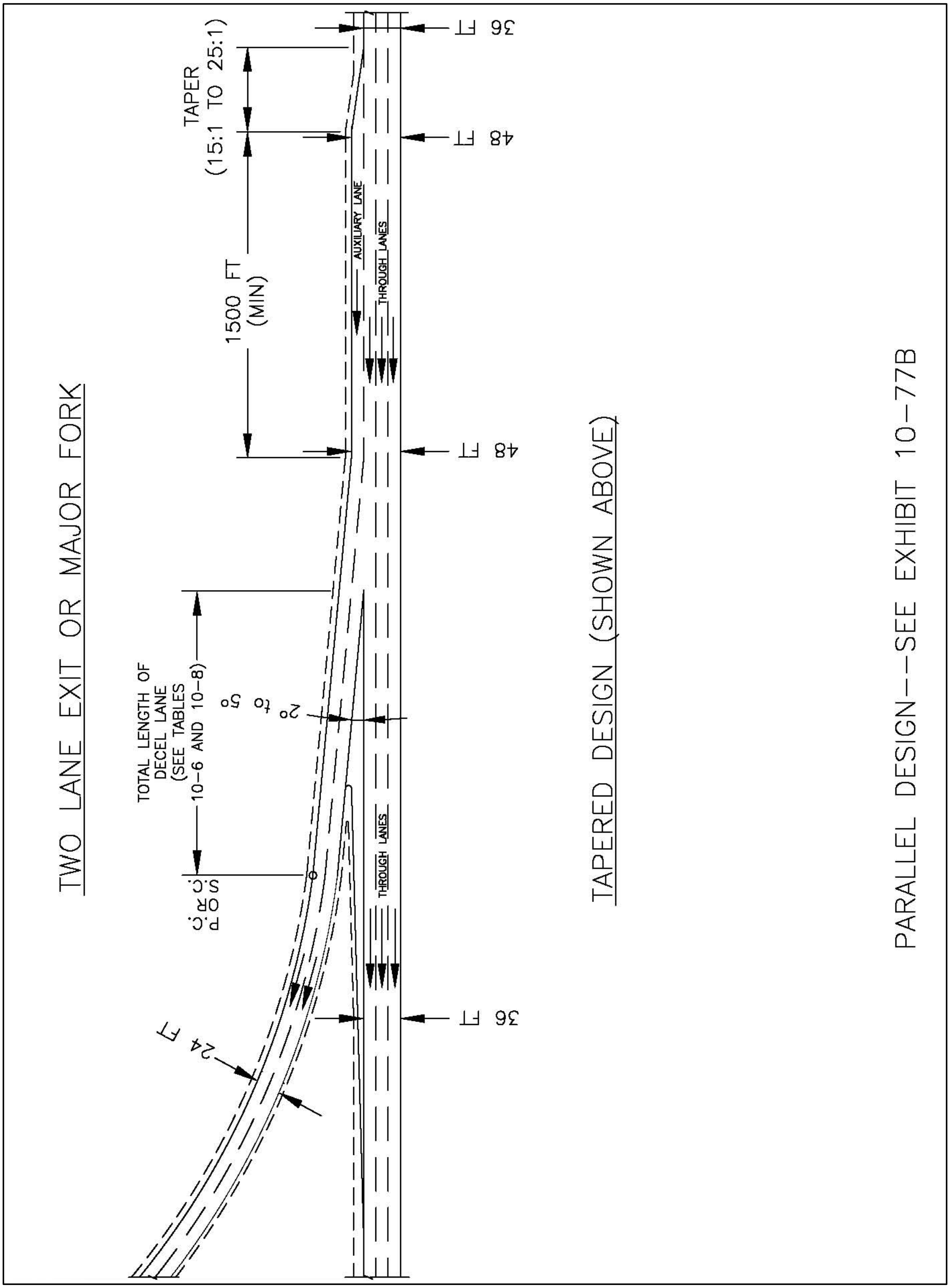


Figure 10-18 Two Lane Exit or Major Fork

On two-lane parallel type exits, the total length from the beginning of the first taper to the point where the ramp traveled-way departs from the right-hand through lane of the freeway should range from 2,500 feet for turning volumes of 1,500 VPH or less upward to 3,500 feet for turning volumes of 3,000 VPH.

If the design year estimated volumes exceed 1,500 equivalent passenger cars per hour (PCPH), a two-lane width of ramp should be provided initially. For volumes less than 1,500 but more than 900 PCPH, a one-lane width exit ramp should be provided with provision for adding an auxiliary lane and an additional lane on the ramp. Provisions may be made for widening to three or even four lanes at the crossroad intersection, depending on the capacity of the intersection. Design of ramp terminals for two-lane exits should conform to the standard designs illustrated by Figure 10-18.

For two-lane exits, the preferred type is the taper type for the same reasons identified previously on the single-lane exit. For the parallel type, traffic in the outer through lanes of the freeway must change lanes twice to exit on the right-hand lane of the exit ramp. This requires considerable lane changing to operate efficiently. Also, the parallel type requires a longer distance from beginning of the taper to the exit nose to develop the full capacity of the ramp.

10.7.9.3 Major Forks and Branch Connections and Freeway-to-Freeway Connections

A major fork is defined as:

- The bifurcation of a directional roadway of a terminating freeway route into two directional multilane ramps that connect to another freeway, or
- The separation of a freeway route into two separate freeway routes of equal importance.

The design of major forks is subject to the same principles of lane balance as any other diverging area. The total number of lanes in the two roadways beyond the divergence should exceed the number of lanes approaching the diverging area by at least one. See Chapter 10 of the PGDHS (1) for additional information.

A branch connection is defined as the beginning of a directional roadway of a freeway formed by the convergence of two directional multilane ramps from another freeway or by the convergence of two freeway routes to form a single freeway route. See Figure 10-76 in the *PGDHS* **(1)**.

10.8 PEDESTRIAN AND BICYCLE ACCOMMODATION

The accommodation of pedestrians and bicycles through urban interchanges should be considered early in the development of interchange configurations. High density land use near interchanges can generate heavy bike and pedestrian movements, resulting in conflicts between vehicles and bicycles or vehicles and pedestrians.

10.9 RAMP METERING

The purpose of ramp metering is to reduce congestion or improve operations on urban freeways. The metering may be limited to only one ramp or integrated into a series of entrance ramps.

Ramp metering consists of traffic signals installed on entrance ramps in advance of the entrance terminal to control the number of vehicles entering the freeway. The traffic signals may be pre-timed or traffic actuated to release the entering vehicles individually or in platoons.

Contact the Region Traffic Engineer for design considerations.

REFERENCES

1. AASHTO. *A Policy on Geometric Design of Highways and Streets*, American Association of State Highway and Transportation Officials, Washington, D.C.: 2011.
2. CDOT Policy Directive 1601.0, Interchange Approval Process January 2004.
3. TRB. *Highway Capacity Manual*, Transportation Research Board, National Research Council, Washington, D.C.: 2010.
4. Caltrans. *Caltrans Highway Design Manual, Sixth Edition.* California Department of Transportation, Sacramento, CA., 2017
5. CDOT. *Standard Plans - M & S Standards*, Colorado Department of Transportation, 2012.
6. AASHTO. *Roadside Design Guide* American Association of State Highway and Transportation Officials, Washington D.C.: 2011

CHAPTER 11
ACCESS CONTROL AND ACCESS MANAGEMENT

11.0 INTRODUCTION

Access management is the planned and regulated interaction between the roadway network and property access. It is an intentional strategy that preserves a safe and efficient transportation system necessary for smart growth. Access management is the practice of limiting and separating conflicts points, and it is the collaboration between planners and developers about property access, and the interaction amongst different modes of travel.

Access management is vital to protect and maintain the public's safety and welfare, maintain the functional roadway classifications, and meet the needs for the traveling public as well as the communities. Access management is a cost-effective approach to ensure the longevity of the public facilities with the likelihood of an increase of population growth and an increase in traffic volumes. Access management strategies must be made with care as these decisions will directly impact adjacent land use.

The objectives of access management are accomplished by applying the following principles found in TRB's *Access Management Manual* **(1)**:

- Provide a specialized roadway system.
- Promote intersection hierarchy.
- Locate signals to favor through movements.
- Preserve the functional area of intersections and interchanges.
- Limit the number of conflict points.
- Separate conflict areas.
- Remove turning vehicles from through traffic lanes.
- Use non-traversable medians on major roadways.
- Provide a supporting street network.
- Provide unified access and circulation systems.

11.1 FUNCTIONAL CHARACTERISTICS AND CATEGORY ASSIGNMENT CRITERIA

The roadway network is such that the highest classification roadways account for the smallest percentage of the network, yet carries the largest percentage of traffic volumes. These highest classification roadways typically carry traffic long distances, and at high speeds. Conversely, the lowest classification roadways account for the highest percentage of total roadway network, carries the least amount of traffic volume (in vehicle miles traveled), and at lower speeds. With this understanding, it is clear to see a direct correlation between roadway functions and land access. Figure 11-1 illustrates this relationship.

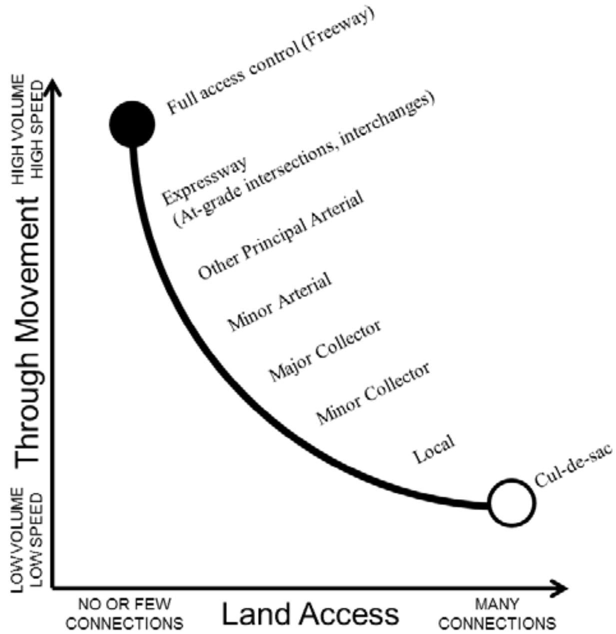


Figure 11-1 [Exhibit 1-1 of the *Access Management Manual* (1)]
Conceptual Roadway Functional Hierarchy

CDOT's *State Highway Access Code* **(2)** defines a system of eight highway categories arranged in a hierarchical order. Table 11-1 illustrates these categories in their tiered order.

F-W Interstate System, Freeway Facilities	
E-X Expressway, Major Bypass	
Rural	**N**on-**R**ural
R-A Regional Highway	**NR-A** Regional Highway
R-B Rural Highway	**NR-B** Arterial
	NR-C Arterial
F-R Frontage Roads (both Urban and Rural)	

Table 11-1 [Table 3.1 of the *State Highway Access Code* (2)] Overview of the Access Category Classification Hierarchy

All segments of all state highways throughout the State of Colorado have been assigned an access category classification, which can be found in the *State Highway Access Category Assignment Schedule* **(3)**. Roadway classifications range from F-W Interstate System, Freeway Facilities as the highest categorized facility, to the F-R Frontage Roads (both urban and rural) as the lowest categorized facility. Descriptions of these functional classifications can be found in Section Three of the *State Highway Access Code*. Also found within this section is criteria for granting access, access spacing, auxiliary lane requirements, and traffic signal treatments.

11.2 ACCESS COORDINATION

Consideration of Access Management is a function of the Operations Analysis within the TSM&O Evaluation. Additionally, coordination and collaboration with stakeholders is a consideration of the Coordination function within the TSM&O Evaluation. See Section 4.12 TSM&O Evaluation in the *Project Development Manual* **(4)**.

Additional information for access points and access permits can be found on CDOT's Access Program Website https://www.codot.gov/business/permits/accesspermits or by contacting the CDOT Access Program Administrator.

11.3 DESIGN STANDARDS AND SPECIFICATIONS

If an access permit is approved, the following design standards and specifications, also found in Section Four of the *State Highway Access Code* **(2)**, shall be followed to meet the criteria defined in Section Three of the *State Highway Access Code* **(2)**.

11.3.1 Sight Distance

Access permits shall only by approved at locations which maintain adequate, unobstructed sight distance in both directions from the access for motorists of the access, or motorists passing the access. See Section 9.8 for intersection sight distances. Table 11-2 identified the appropriate design vehicle to be used for sight distance calculations.

Land Use(s) Served by Access	**Design Vehicle(s) (to be used for Sight Distance calculations)**
Residential (A Non-School Bus Route)	Passenger Cars, Pickup Trucks
If Access is a Part of Any School Bus Route Regardless of Land Use	No Less Than Single Unit Trucks
Office	Single Unit Trucks
Recreational	Single Unit Trucks
Commercial/Retail	Multi-Unit Trucks*
Industrial	Multi-Unit Trucks*
Municipal Streets & County Roads	Multi-Unit Trucks*
Agricultural Field Approaches, < 1 Per Day	Single Unit Trucks
*If Less Than 2 Multi-Unit Truck Trips Per Day (Average), Use Single Unit Truck	

Table 11-2 [Table 4-3 of the *State Highway Access Code* (2)] Design Vehicle Selection

11.3.2 Access Spacing

The minimum spacing between accesses is based on the calculated sight distance along the highway. In instances where speed change lanes are or will be present, access spacing shall be a minimum of the speed change lane including transition tapers. Accesses shall not be permitted within a speed change lane including transition tapers. See section 9.17 for speed change lane requirements.

11.3.3 Access Width

Table 11-3 illustrates access widths for one-way and two-way accesses. In instances where a public roadway access intersects the state highway, the access width shall be a function of long term traffic projections and modal use.

<table>
<tr><th colspan="2">Design</th><th>Criteria</th></tr>
<tr><td>One-Way Access</td><td>16' - 18'</td><td></td></tr>
<tr><td rowspan="2">Two-Way Access</td><td>16' - 30'</td><td>SU Peak Hour Volume < 5</td></tr>
<tr><td>25' - 40'</td><td>When one or more of the following apply:
• SU Peak Hour Volume > 5
• Multi-Unit Vehicles intended to use Access
• SU Vehicles in excess of 30 feet in length
• Special Vehicles Using the Access > 16 feet wide</td></tr>
<tr><td>Two-Way Public Access</td><td>> 36'</td><td>Design Hourly Volume > 10</td></tr>
</table>

Table 11-3 Access Width

See the *State Highway Access Code* **(2)** for additional access width measurement criteria.

11.3.4 Access Radii

Access radii shall be a minimum of 20 feet. In instances where shoulders are not present, access radii shall be 25 feet for residential and field accesses. If the design vehicle intended to use the access daily is a single-unit exceeding 30 feet, multi-unit, or another vehicle requiring a larger radius, the minimum turn radius accommodating this design vehicle shall be used. Access radii shall allow safe maneuvers without intrusion into adjacent highway travel lanes. In instances where multiple larger vehicles are likely to oppose each other at the access, the radii should be adequate to accommodate both vehicles without conflict or undue slowing. Local design standards shall be followed unless minimums listed here are not met. Radii shall be designed only to that required to minimize pedestrian conflicts.

See the *State Highway Access Code* **(2)** for additional access radii information.

11.3.5 Access Surfacing

Accesses shall be surfaced before opening to public use. Access surface material may include gravel, asphalt, and concrete. At a minimum, accesses shall be surfaced between the roadway and the right-of-way line. Table 11-4 illustrates the hard surface minimum limits. In instances where a hard surface access joins existing pavement, a minimum one foot saw cut is required for the tie in. Access surfacing materials and design shall conform to the local design standards, unless minimums listed here are not met.

Criteria	Hard Surface Minimum Limits From Traveled Way (ft)
5 AADT	4
20 AADT	20
100 AADT	50
Turn Lane	50

Table 11-4 Hard Surface Minimum Limits

See the *State Highway Access Code* **(2)** for additional access surfacing information.

11.4 SPEED CHANGE LANES

Speed change lane considerations are discussed in section 9.17.

11.5 OTHER DESIGN ELEMENTS

At curb cut locations, crest curves shall not exceed a four inch hump per ten foot chord, and sag curves shall not exceed a four inch dip per ten foot chord, to assist in preventing vehicle drag. At locations which utilize curb returns and not curb cuts, the first 20 feet beyond the travel way shall slope away from the highway at two percent. Some exceptions may be permitted on a case by case basis, but shall protect the highway from drainage flows.

Within the right-of-way, field and residential accesses shall not exceed ten percent grade, and all other accesses shall not surpass eight percent. Accesses within the right-of-way shall be designed to not impede future use of the right-of-way. The access centerline shall intersect the highway centerline at 90 degrees unless safety concerns dictate otherwise. The access shall extend from the edge of travel way in a tangent direction at least 40 feet, or to the right-of-way, whichever is greater.

All signing, striping, traffic signals, and other traffic control, shall conform to standards presented in the *Manual on Traffic Control Devices (MUTCD)* **(5)**.

See the *State Highway Access Code* **(2)** for additional design information.

11.6 EMERGENCY ACCESS

Emergency accesses may be less than 16 feet wide, so long as one-way traffic is still accommodated. The access should be unsuspecting to avoid use by the public but still of sufficient strength to accommodate emergency vehicles. Radii may be omitted as emergency vehicles may utilize the access unimpeded. Any barrier used to close off this access must be outside of the highway right-of-way.

See the *State Highway Access Code* **(2)** for additional emergency access information.

11.7 DRAINAGE

The existing highway drainage system is designed to accommodate the drainage relative to the state highway and not for development outside of the right-of-way (beyond historical flows). Any drainage entering the system from accesses shall not exceed the rate of historical flow. Any drainage appurtenances such as a detention pond must be fully located outside of the right-of-way. In locations where curb and gutter exist, a storm sewer system should be the drainage option of design. In locations where there is no curb and gutter, a roadside ditch should be the drainage option of design.

See the *State Highway Access Code* **(2)** for additional drainage information.

REFERENCES

1. TRB. *Access Management Manual*, Second Edition, Transportation Research Board, Washington D.C.: 2014.
2. CDOT. *State Highway Access Code (Code of Colorado Regulations 601-1),* as adopted and amended by the Transportation Commission of Colorado, Revised March 2002.
3. CDOT. *State Highway Access Category Assignment Schedule, (Code of Colorado Regulations 2 CCR 601-1A)*, as revised and adopted by the Transportation Commission of Colorado, September 2013.
4. CDOT. *CDOT Project Development Manual*, Colorado Department of Transportation, 2013 (with revisions through 2016).
5. FHWA. *Manual on Uniformed Traffic Control Devices for Streets and Highways (MUTCD)*, U.S. Department of Transportation, Federal Highway Administration, Washington, D.C.: 2009.

List of Definitions

Accessible - Describes a facility in the public right-of-way that complies with the requirements set forth in the ADAAG or PROWAG.

ADA - An acronym for the Americans with Disabilities Act.

ADA Transition Plan - The 1991 ADA regulation (28 CFR Part 35) required all public entities to evaluate all of their services, policies, and practices and to modify any that did not meet ADA requirements. Public entities with 50 or more employees were required to develop a transition plan detailing any structural changes that would be undertaken to achieve program access and specify a period for their completion.

ADAAG – An acronym for ADA Accessibility Guidelines

Accessible Pedestrian Signal (APS) - A traffic signal that incorporates a device to communicate information to a pedestrian in audible and vibrotactile formats.

Alteration - A change to a facility in the public right-of-way that affects or could affect pedestrian access, circulation, or use. Alterations include, but are not limited to, resurfacing, rehabilitation, reconstruction, historic restoration, or changes or rearrangement of structural parts or elements of a facility.

Blended Transition - A raised pedestrian street crossing, depressed corner, or similar connection between the pedestrian access route at the level of the sidewalk and the pedestrian street crossing. Blended transitions have a grade of five percent or less.

Cross Slope - The grade that is perpendicular to the dominant direction of pedestrian travel.

Curb Ramp - A combination of a ramp and landing to accomplish a change in elevation at a curb face. This element provides street and sidewalk access to pedestrians in wheelchairs or with mobility impairments.

Detectable Warning Surface (DWS) - A standardized surface feature of truncated domes that provides an indication to individuals with disabilities that they are transitioning from the pedestrian realm to the vehicular way.

Equivalent Facilitation - The use of alternative designs, products, or technologies that result in substantially equivalent or greater accessibility and usability than the requirements in the PROWAG and/or ADAAG.

Facility - All or any portion of buildings, structures, improvements, elements, and pedestrian or vehicular routes located in the public right-of-way.

Grade Break - The line where two surface planes with different grades meet.

Landing - The area of the Pedestrian Access Route at the top or bottom of a curb ramp. Also referred to as "Turning Space".

Maximum Extent Feasible - Where existing physical constraints make it impracticable for altered elements, spaces, or facilities to fully comply with the requirements for new construction, compliance is required to the extent practicable within the scope of the project. Also referred to as "Maximum Extent Practicable".

Mobility Device – Any assistive technology or device that aids the movement of people with a physical impairment. Examples include lift chairs, wheelchairs, scooters, etc.

Operable Part - A component of an element used to insert or withdraw objects, or to activate, deactivate, or adjust the element.

Other Power-Driven Mobility Device (OPMD) – An OPDMD is any mobility device powered by batteries, fuel, or other engines that is used by individuals with mobility disabilities for the purpose of locomotion, whether or not it was designed primarily for use by individuals with mobility disabilities. OPDMDs may include electronic personal assistance mobility devices, such as the Segway ® Personal Transporter (PT), or any mobility device that is not a wheelchair and is designed to operate in areas without defined pedestrian routes.

Pedestrian - A person afoot or in a wheelchair or other power-driven mobility device (OPMD).

Pedestrian Access Route (PAR) - A continuous and unobstructed path of travel provided for pedestrians with disabilities within or coinciding with a pedestrian circulation path.

Pedestrian Circulation Path - A prepared exterior or interior surface provided for pedestrian travel in the public right-of-way.

PROWAG - An acronym for the Public Rights-of-Way Accessibility Guidelines.

Qualified Historic Facility - A facility that is listed in or eligible for listing in the National Register of Historic Places, or designated as historic under an appropriate state or local law.

Running Slope - The grade that is parallel to the dominant direction of pedestrian travel.

Sidewalk - A paved pathway paralleling the highway or street that is intended for pedestrian travel.

Shared Use Path - A travel-way separated from motor vehicle traffic by an open space or barrier and either within the Right-of-Way or in an independent Right-of-Way. Shared use paths may be used by pedestrians, bicyclists, and other non-motorized users. See Chapter 14 for more information on Shared Use Path Design.

Technically Infeasible - With respect to an alteration of a building or facility, something that has little likelihood of being accomplished because existing structural conditions would require removing or altering a load-bearing member that is an essential part of the structural frame; or because other existing physical or site constraints prohibit modification or addition of elements, spaces, or features that are in full and strict compliance with minimum requirements.

Turning Space - (see “Landing”)

Vertical Surface Discontinuities - Vertical differences in level between two adjacent surfaces.

Vibrotactile – Relaying information to the user through the perception of vibration or touch.

12.0 INTRODUCTION

Pedestrians may be present in all roadway environments. Pedestrian access, accommodation, and safety should be given full consideration during all project planning, scoping, and design. Accommodation can take many forms but most often appears as sidewalks or shared use paths. In rural areas or on roadways with limited pedestrian demand, paved shoulders and sometimes no facility at all may be an acceptable level of pedestrian accommodation. Sidewalks are not a requirement on every roadway; however, if there is evidence that pedestrian demand exists suitable pedestrian facilities shall be provided.

The Americans with Disabilities Act of 1990 (ADA) was the nation's first comprehensive civil rights law and prohibits discrimination against people with disabilities. Under Title II of the ADA and Section 504 of the Rehabilitation Act of 1973 (504), entities that are responsible for roadway and pedestrian facilities may not discriminate on the basis of disability in any transportation program, activity, service, or benefit which is provided to the general public. In other words, entities responsible for transportation infrastructure must ensure that people with disabilities have equitable opportunities to use the infrastructure that is provided to the public. The ADA accessibility requirements apply throughout the entire transportation facility lifecycle including planning, design, construction, maintenance, and operation.

Where pedestrian facilities are provided, they must be constructed so they are accessible to all potential users. This Chapter provides guidance and direction regarding the requirements for the design of accessible pedestrian facilities.

Under the ADA, the Department of Transportation (DOT) and the Department of Justice (DOJ) are responsible for issuing and enforcing accessibility standards. The standards are developed by the US Access Board, which is an independent agency that promotes equality for people with disabilities and is the leading source of information on accessible design. The US Access Board's guidelines become enforceable once they are adopted by the respective standard setting agency, which in the case of transportation facilities is the DOT.

To ensure that people with disabilities have access to the built environment the US Access Board developed design guidelines known as the ADA Accessibility Guidelines (ADAAG). However, pedestrian infrastructure in the public right-of-way can pose many challenges to accessibility, and the ADAAG was developed with a focus on providing accessible buildings and facilities. Although the ADAAG does address some features found within the public right-of-way it was determined that additional guidance was necessary to address conditions and constraints that are unique to public rights-of-way. The additional guidance developed was titled the Public Rights-of-Way Accessibility Guidelines (PROWAG). These draft guidelines are not standards until adopted by the DOJ and the DOT. However, the PROWAG is currently recommended as best practice by the FHWA (January 2006 FHWA memo) and can be considered the state of practice for areas not fully addressed by the ADAAG.

Both the ADAAG and the PROWAG provide the means to meet the requirements of the ADA. Generally, facilities located within the public right-of-way should be consistent with the requirements set forth in the PROWAG. Once the PROWAG is adopted by the DOJ and the DOT it becomes the enforceable standard for transportation facilities under Title II of the ADA. The guidance in this chapter is based upon the PROWAG requirements, and references to the PROWAG document are shown in parentheses (for example - R202.3).

More information on the ADAAG and the PROWAG can be found at:

https://www.access-board.gov/guidelines-and-standards/buildings-and-sites/about-the-ada-standards/ada-standards

https://www.access-board.gov/guidelines-and-standards/streets-sidewalks

12.1 ADA ACCESSIBILITY REQUIREMENTS, STANDARDS & GUIDELINES

The ADAAG and the PROWAG are not requirements of the ADA, but serve as the standards and guidelines by which compliance of the law is measured. Generally, the ADA law requires:

- New construction to be accessible
- Alterations to existing facilities that are within the scope of a project to provide accessibility to the maximum extent feasible
- Existing facilities that have not been altered shall not deny access to persons with disabilities

New Construction Project Requirements

All new construction projects where a pedestrian demand is exhibited shall incorporate appropriate pedestrian facilities that are accessible to persons with disabilities. New construction projects have the ability to mitigate constraints through good planning and design practices. Project budget or limited scopes are not an acceptable reason to fail to provide compliant accessible facilities during new construction.

Alteration Project Requirements

Whenever existing facilities are altered, each altered element must meet the most current accessibility standard if it is within the scope of the alteration project. If elements are within a project's limits and scope but are not accessible, they must be made so. For example, if a project is resurfacing a roadway and curb ramps are missing at pedestrian crossings, compliant curb ramps must be incorporated into that project because that project affected the crossing served. That same project would not be required to install pedestrian push buttons if the existing signals are to remain, because that work is outside of the original resurfacing scope.

Where existing physical constraints make it impractical for facilities to comply with the current standard, improvements must be made to provide accessibility to the maximum extent practical within the scope and limits of the project. Alteration projects shall not intentionally skip or "gap" pedestrian elements to avoid triggering the requirement to make ADA improvements.

Only the altered portion of a facility is required to be made compliant at the time of a project. If elements are altered or added but the pedestrian circulation path is not altered, the pedestrian circulation path is not required to be made accessible. However, it is often beneficial to improve surrounding unaltered facilities while construction forces are mobilized. When possible, consideration should be given to making nearby facilities accessible. The alteration of a facility may affect the usability of an adjoining facility and additional improvements can at times be unavoidable. An alteration project shall not decrease accessibility, or the accessibility to a connecting or adjacent building or site, below the current standards required of new construction during the time of the alteration (R202.3). These considerations should be taken into account during project scoping.

If the State Historic Preservation Office (SHPO) determines that an alteration that is required for compliance would threaten or destroy historically significant features of a qualified facility, improvements shall be made to the extent that they do not destroy the historically significant features of that facility (R202.3.4).

All alteration projects must remove existing pedestrian access barriers. For example, projects must install curb ramps at locations where they are missing if they are within the limits of the altered area.

Examples of Alteration projects include, but are not limited to:

- Addition of new layers of Asphalt
- Mill and Fill / Mill and Overlay
- Rehabilitation & Reconstruction
- Cape Seals
- Microsurfacing and thin lift overlays
- Widening
- Bridge projects

For more information regarding what treatments or combinations thereof constitute alterations see the DOT's technical assistance information (links below). The DOT has prepared the graphic below to help illustrate what items may be considered maintenance versus alterations. If two maintenance items are performed at the same time, they may rise to the level of an alteration. The technical assistance from the DOT should be consulted when determining whether an item(s) is considered maintenance or an alteration.

1. https://www.fhwa.dot.gov/civilrights/programs/doj_fhwa_ta.cfm
2. https://www.fhwa.dot.gov/civilrights/programs/doj_fhwa_ta_glossary.cfm
3. https://www.fhwa.dot.gov/civilrights/programs/ada_resurfacing_qa.cfm

Non-Alteration Project Requirements

Activities that are considered normal maintenance do not require simultaneous improvements for accessibility under the ADA. The DOT has offered the following guidance regarding what kinds of treatments constitute regular maintenance activities as opposed to an alteration:

"*Treatments that serve solely to seal and protect the road surface, improve friction, and control splash and spray are considered to be maintenance because they do not significantly affect the public's access to or usability of the road. Some examples of the types of treatments that would normally be considered maintenance are: painting or striping lanes, crack filling and sealing, surface sealing, chip seals, slurry seals, fog seals, scrub sealing, joint crack seals, joint repairs, dowel bar retrofit, spot high-friction treatments, diamond grinding, and pavement patching**. In some cases, the combination of several maintenance treatments occurring at or near the same time may qualify as an alteration and would trigger the obligation to provide curb ramps."*** **(Emphasis added)**

Generally, maintenance activities are considered those actions that are intended to preserve the functionality or condition of an asset without increasing its capability or structural capacity.

Emergency repairs, such as interim paving or patching, would not trigger the requirement to upgrade or install accessible pedestrian facilities.

Technically Infeasible

It can be impractical to make facilities fully compliant with the standards due to existing site constraints. Improvements at locations can be deemed “Technically Infeasible” when sound engineering judgement is exercised. When full compliance is deemed technically infeasible, facilities being altered should be made accessible to the maximum extent practicable. If a site cannot meet accessibility standards, the proper documentation procedures should be followed. For more information, visit CDOT’s Civil Rights ADA Resources for Civil Engineers webpage. For example, should a hypothetical curb ramp be constrained by a historic property, CDOT staff should visit the Civil Rights ADA Resources for Civil Engineers webpage and download the **Curb Ramp Variance Support Document** in order to prepare documentation for the affected ramp. Curb Ramp Variance Support Documents should be initiated as early as possible on projects so one ramp does not hinder the project schedule.

Examples of site constraints that may make it technically infeasible to make a facility fully compliant include:

- Underground structures or utilities that would have to be altered to make a facility compliant and would expand the project scope.
- Adjacent development or buildings that would need to be moved or altered to make a facility fully compliant.
- Required improvements that would alter the status of a Historic property.
- Drainage that could not be maintained if an area is made fully accessible.
- Underlying terrain that would require significant expansion of the project scope to achieve full compliance. An example would be altering a roadway profile to make the cross slope of a crosswalk fully compliant.

Project scope, not cost, should determine when existing constraints make an item technically infeasible. For example, a resurfacing project that does not include the relocation of existing utilities may be justified in providing a facility that does not comply fully with the standards and is accessible to the maximum extent feasible if those utilities prohibit compliance. However, a widening project that includes right-of-way acquisition and utility relocations would not be able to use that same justification because those elements are within the scope of the project.

Unaltered Existing Facilities & Transition Plan

Facilities that are not to be addressed in any current or planned CDOT projects are to be addressed through CDOT’s ADA Transition Plan. For more information about the plan and to access the plan itself visit https://www.codot.gov/business/civilrights/ada/transition-plan

12.2 TECHNICAL REQUIREMENTS FOR ACCESSIBILITY

The following section provides detailed technical criteria and guidance for the development of accessible pedestrian routes. The material in this section is derived from information found in the PROWAG and other relevant sources. It is the intent of this Chapter to be consistent with all of the criteria provided in existing federal or CDOT standards. This section is intended to provide the most relevant technical requirements in one location and provide additional guidance and best practices when possible. The minimum and maximum values that are provided are taken from the PROWAG. Target values may also be provided. **Designing features to values other than the allowable minimums or maximums allows for adjustments in the field and provides flexibility during construction.** This practice is encouraged when possible.

Pedestrian Access Route Technical Requirements

A pedestrian access route (PAR) is a continuous and unobstructed path of travel intended to provide accessibility for pedestrians with disabilities. A pedestrian access route shall be provided where a prepared surface has been constructed for pedestrian travel within the right-of-way. Examples of areas that may be considered a PAR include:

- Crosswalks at intersections
- Curb ramps
- Pedestrian overpasses and underpasses
- Sidewalks
- Shared-use paths
- Elevators
- Doorways
- Parking access aisles.

The following describes the common requirements of the PAR.

Continuous Width (R302.3) - The continuous width of the PAR shall be 4 feet minimum, exclusive of the curb. Where a pedestrian access route makes a 90 degree turn, it should be widened to 5 feet to accommodate the continuous passage of a wheelchair (i.e. pedestrian design vehicle). CDOT projects should provide 5-foot sidewalks unless unique constraints are present. If the clear width of the PAR is less than 5 feet, passing spaces shall be provided at a maximum of 200-foot intervals. If passing spaces are provided they shall be 5 feet by 5 feet minimum. The clear width of a pedestrian refuge island shall be 5 feet minimum.

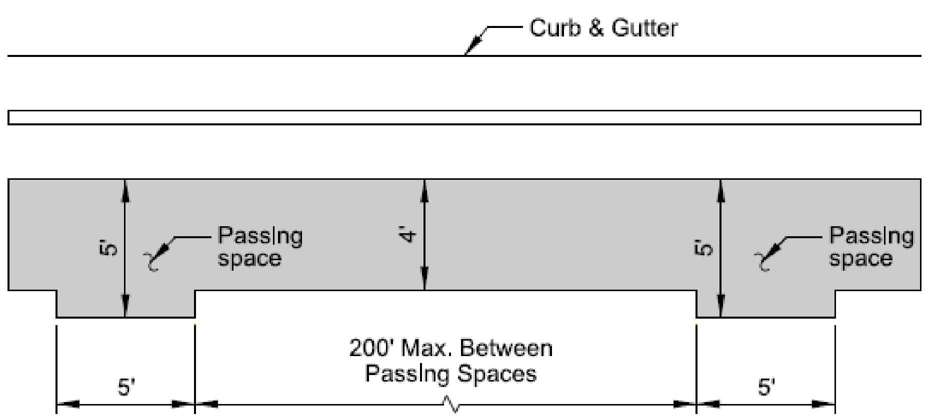


Figure 12-1 - Passing Spaces

Pinch Points - Pinch points within the PAR shall not be less than 34 inches in width and not exceed 24 inches in the direction of pedestrian travel. A study by the U.S. Access Board determined that a 34 inch width allowed 95% of the wheel mobility users to pass the obstruction in the study areas (Anthropometry of Wheel Mobility Project, US Access Board). Pinch Points are permitted in constrained areas on maintenance and alteration projects only with approval of the Project Engineer. **Pinch points are not acceptable in new construction.**

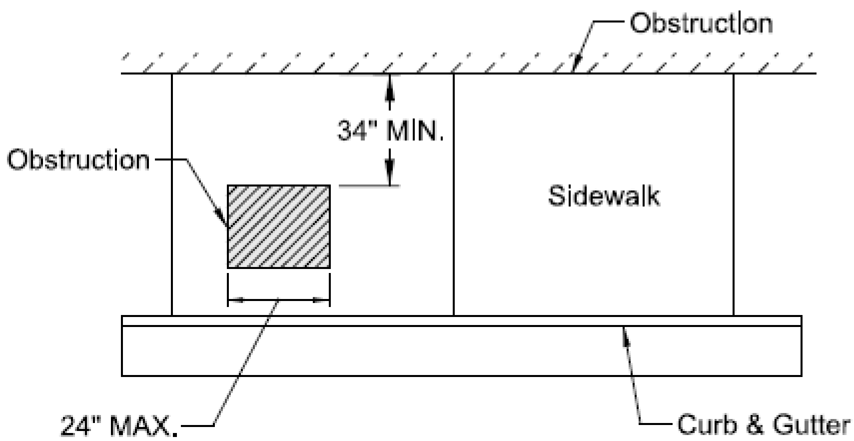


Figure 12-2 - Pinch Points

Surface (R302.7) - The surface of the PAR must be firm, stable, and slip resistant. A standard test has not been defined to measure slip resistance or stability, so sound engineering judgement must be applied to ensure that users with mobility impairments can traverse the PAR. The preferred surface for pedestrian walkways and sidewalks is concrete with a transverse broom finish. Examples of noncompliant surfaces include cobblestones, split-faced stone, loose sand, dirt, gravel, and any other similar irregular surface. Grade breaks along the PAR and within curb ramps shall be flush. The characteristics of the surface when wet should be taken into consideration when determining if a surface is firm, stable, and slip resistant.

There exists an allowance for vertical surface discontinuities for occasional expansion joints and objects such as utility covers, vault frames, and gratings that cannot be located in another portion of the sidewalk or outside the PAR. Objects such as utility covers, vault frames, and gratings should not be located on curb ramp runs, blended transitions, turning spaces, or gutter areas within the PAR. This may not always be possible in alterations, but should be avoided when possible. Vertical surface discontinuities between unit pavers should be minimized. Vertical surface discontinuities shall be a 0.5-inch maximum. When there is a surface discontinuity between 0.25 inch and 0.5 inch, the discontinuity shall be beveled at 2:1. The bevel shall be provided for the entire length of the discontinuity.

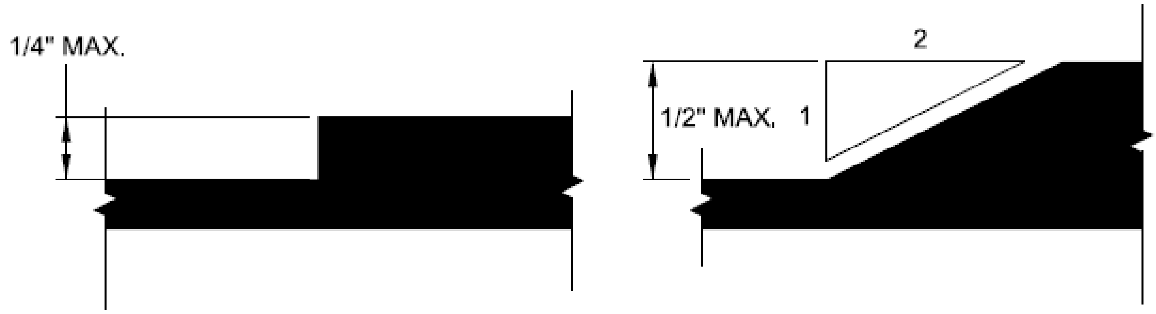


Figure 12-3 - Allowable Vertical Discontinuities

Where the PAR crosses a rail at grade, the pedestrian access route surface shall be level with the top of the rail. The surface between the rails shall be level with the top of the rails.

Horizontal Openings & Horizontal Gaps (R302.7.3) - The horizontal openings in grates shall not permit passage of a sphere more than 0.5 inch in diameter. Grate openings should be placed so that the long dimension is perpendicular to the direction of travel of pedestrians. The use of grates and inlets within the PAR should be avoided.

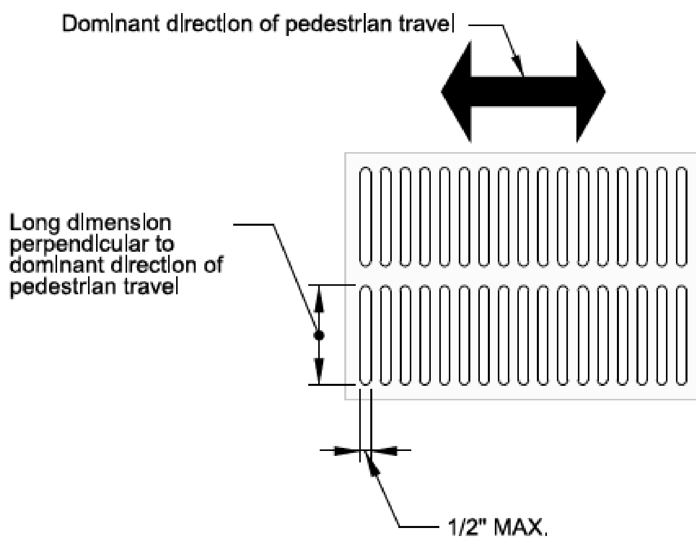


Figure 12-4 - Horizontal Openings in Grates

When the PAR crosses a railway the horizontal gap between the pedestrian surface and the rail shall be no more than 2.5 inches on a non-freight railway and no more than 3 inches on a freight railway.

Protruding Objects (R402) - Objects which are between 2.25 feet (27 inches) and 6.7 feet (80 inches) above the finished surface of the walkway shall not protrude into the pedestrian access route more than 4 inches.

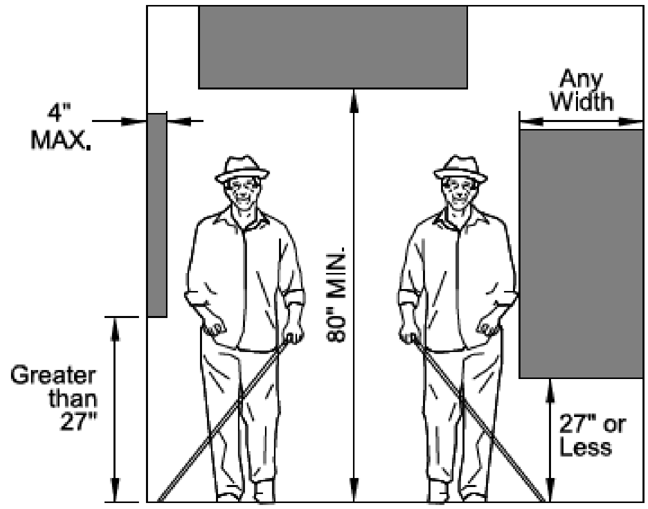


Figure 12-5 - Protruding Objects

When vertical clearance is less than 6.7 feet (80 inches), a guardrail or barrier to pedestrian travel shall be provided. The leading edge of the barrier or guardrail shall not be more than 2.25 feet (27 inches) above the finished surface.

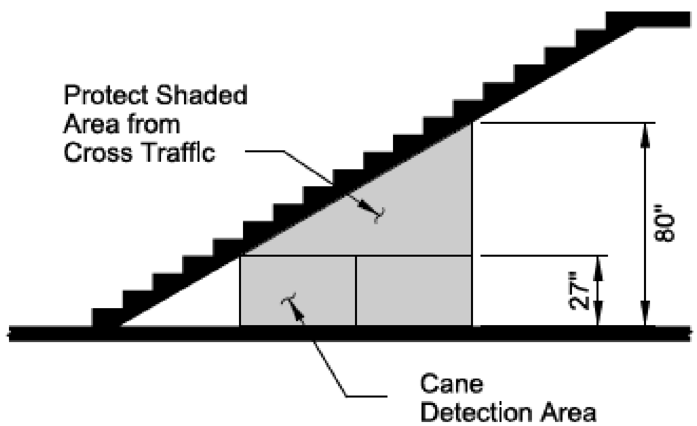


Figure 12-6 - Protection from Stairways

Grade (R302.5) - When the PAR is contained within the street or highway right-of-way the grade of the PAR can match the general grade of the street or highway. When the PAR is not within the street or highway right-of-way the grade of the PAR shall not be greater than 5.0%. The grade of the pedestrian access route is measured parallel to the dominant direction of pedestrian travel.

Cross Slope (R302.6) - The cross slope of the PAR is measured perpendicular to the dominant direction of travel and shall not exceed 2.0%.

The PROWAG has set limits for the maximum allowable cross slope at pedestrian street crossings. At these locations the roadway longitudinal grade becomes the cross slope of the pedestrian crossing. The maximum allowable cross slope for pedestrian street crossings is dependent on the type of vehicular traffic control present at the crossing. At times, these requirements limit the longitudinal grade of the roadway and require a “tabled crosswalk” at the intersection. Pedestrian crossings without yield or stop control are defined as those without a yield or stop sign, or where a traffic signal is designed to remain in the green phase. The following are the requirements for cross slope of pedestrian street crossings:

1. *Intersection leg with yield or stop control* - The pedestrian street crossing must not have a cross slope greater than 2.0%.
2. *Intersection leg without yield or stop control* - The pedestrian street crossing must not have a cross slope greater than 5.0%.
3. *Mid-block pedestrian crossing* - The pedestrian street crossing is allowed to have a cross slope equal to the street or highway grade.

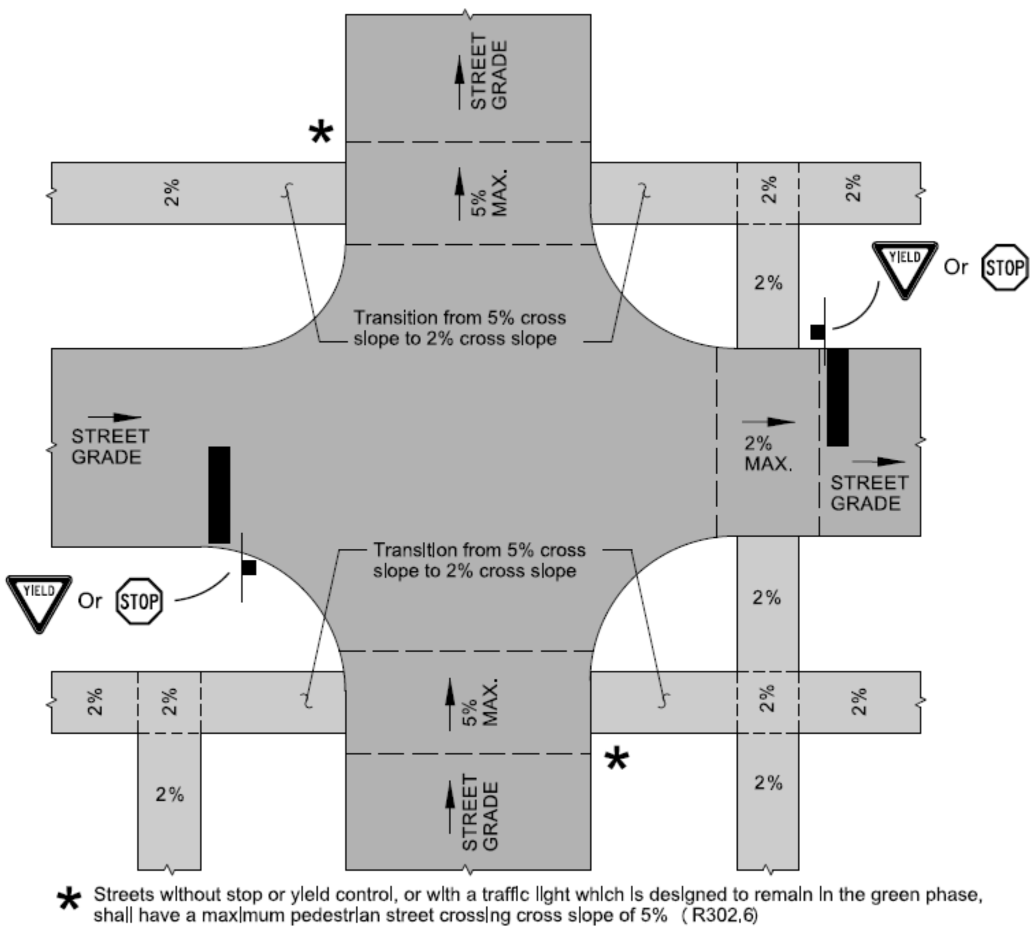


Figure 12-7 - Pedestrian Street Crossing Cross Slope

12.3 CURB RAMP GENERAL INFORMATION

Curb ramps are intended to provide pedestrians access between the sidewalk and street when a curb face or vertical change in elevation is present. For new construction or on streets being altered, curb ramps are mandated by Title II of the ADA. In addition to providing access for those with mobility impairments, curb ramps also make street crossings easier for pedestrians without disabilities including people pushing strollers, riding bicycles, and making deliveries.

Most curb ramps contain a combination of the following core elements: approach, ramp runs, flares, vertical curb faces, landings or turning spaces, transition between the ramp run and gutter, and detectable warnings. These common elements can be configured in several ways to create a variety of curb ramp designs. Generally, curb ramps can be grouped into three categories: perpendicular ramps, parallel ramps, and blended transitions.

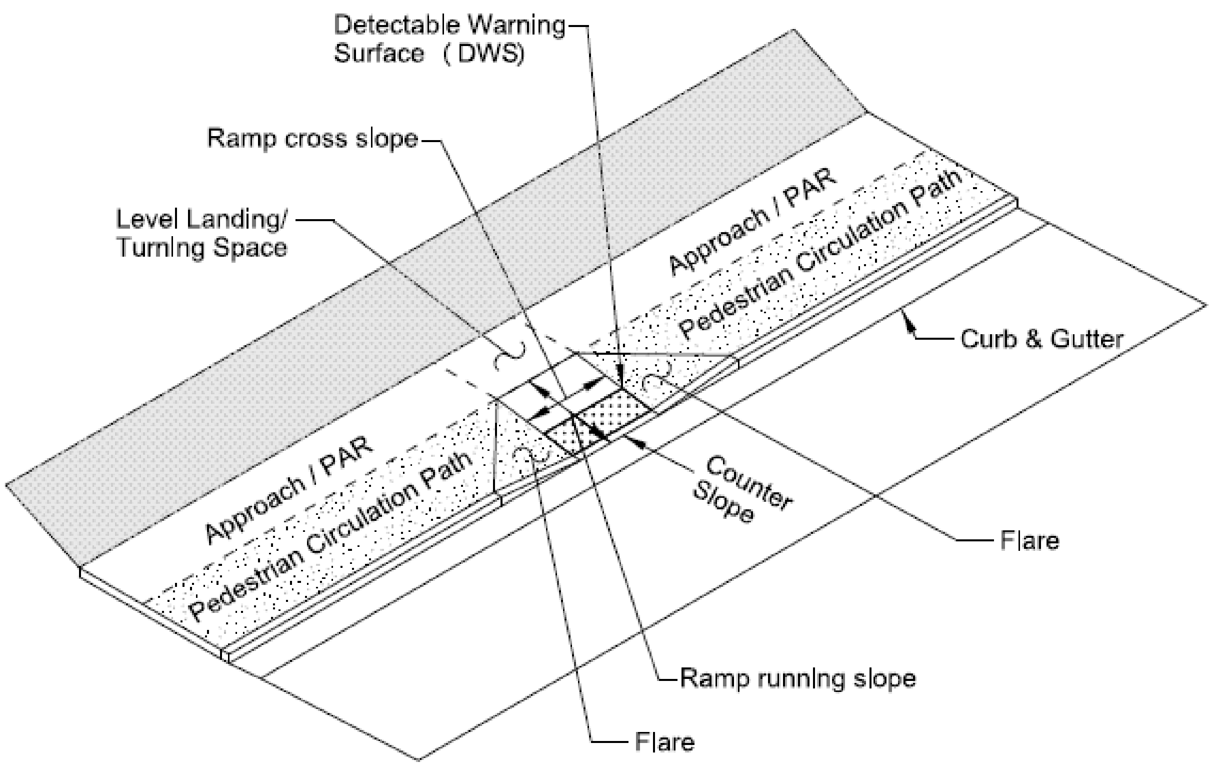


Figure 12-8 - Curb Ramp Elements

Curb ramps should not be placed in locations where pedestrians must cross drainage inlets or ponding water. Designers should consider the location of drainage inlets relative to any curb ramps that are being installed. Locating or moving drainage inlets upstream of curb ramps helps eliminate ponding and should be a consideration during design. Curb ramps should be located away from the low point of a curb return when possible.

Often curb ramp locations will be determined by the existing site constraints, however, the preferred design is to have a separate curb ramp aligned with each crossing. In new construction, where existing constraints do not exist, one curb ramp for each crossing is required. On CDOT projects, a single diagonal curb ramp (on the apex) will only be permitted on reconstruction and alteration projects where physical site constraints prevent two curb ramps from being installed. Designers should make every attempt to provide one ramp per crossing.

Diagonal curb ramps present several challenges of which designers should be aware:

- Providing a level clear space at the bottom of a ramp is often difficult to achieve.
- Diagonal ramps present a problem for pedestrians because pedestrians are directed towards the middle of the intersection. This may be particularly troublesome for pedestrians with vision impairments who cannot determine the correct alignment of the street crossing.
- They create uncertainty for motorists who cannot determine which direction the pedestrian is trying to cross. As a result, motorists are less likely to yield to pedestrians trying to cross the street.

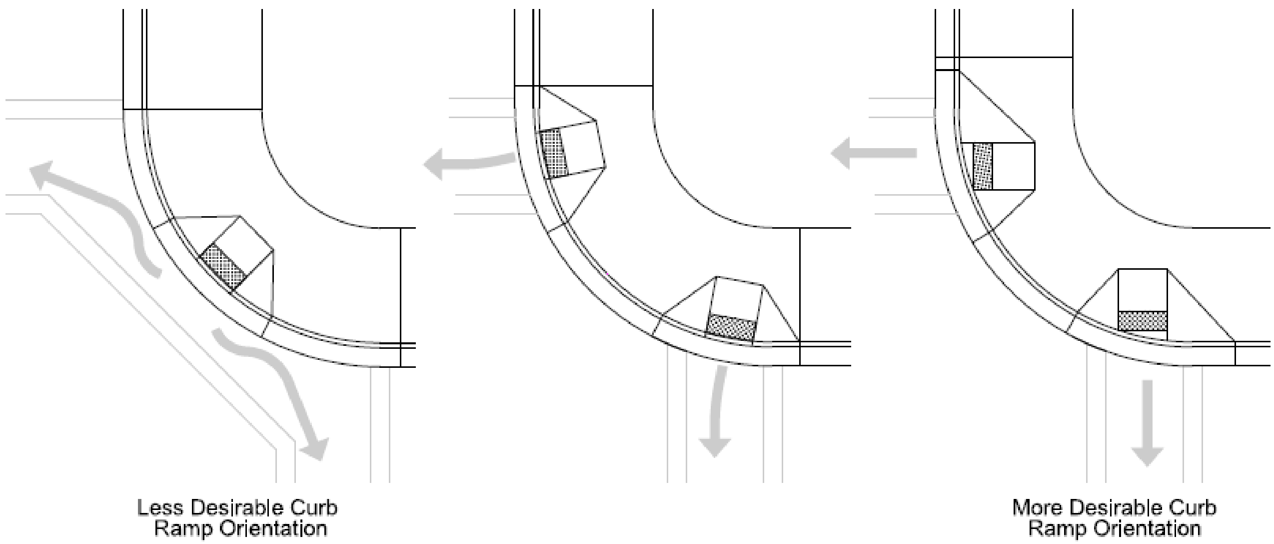


Figure 12-9 - Preferred Curb Ramp Placement

12.4 CURB RAMP TYPES

Perpendicular curb ramps are oriented perpendicular to the curb face or vertical elevation change they traverse. Perpendicular ramps have a turning space located at the top of the ramp to allow users to get oriented in the direction of the crossing before travelling down the ramp. Perpendicular curb ramps are generally the preferred design to accommodate pedestrians if there is enough space for their installation. When possible, perpendicular ramps should be aligned with the pedestrian street crossing they serve. When space is not available for the installation of a perpendicular curb ramp consideration should be given to the use of parallel curb ramps.

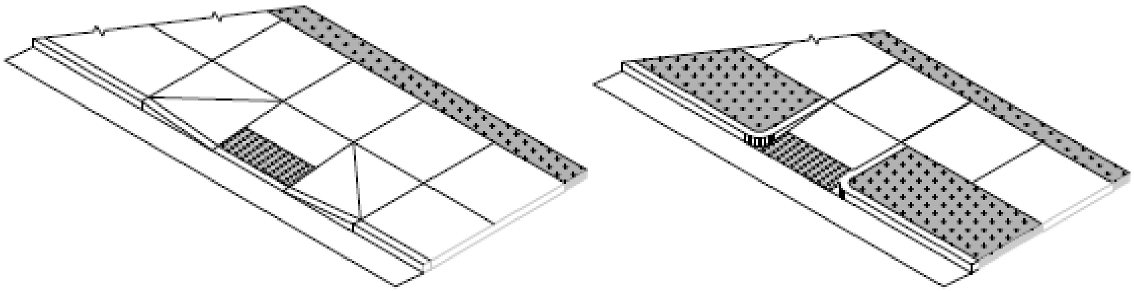


Figure 12-10 - Perpendicular Curb Ramp Examples

Parallel curb ramps are oriented parallel to the curb face or elevation change which they traverse. These are often used when there is little room between the curb and the back of sidewalk for a perpendicular curb ramp and turning space. Parallel curb ramps transition down to the roadway elevation and require individuals to traverse multiple ramp surfaces when traveling along a sidewalk. To avoid this, it is preferred that perpendicular ramps be used where possible. Additionally, because the turning space of a parallel ramp is at the roadway grade, sedimentation and drainage can be an issue with parallel style curb ramps.

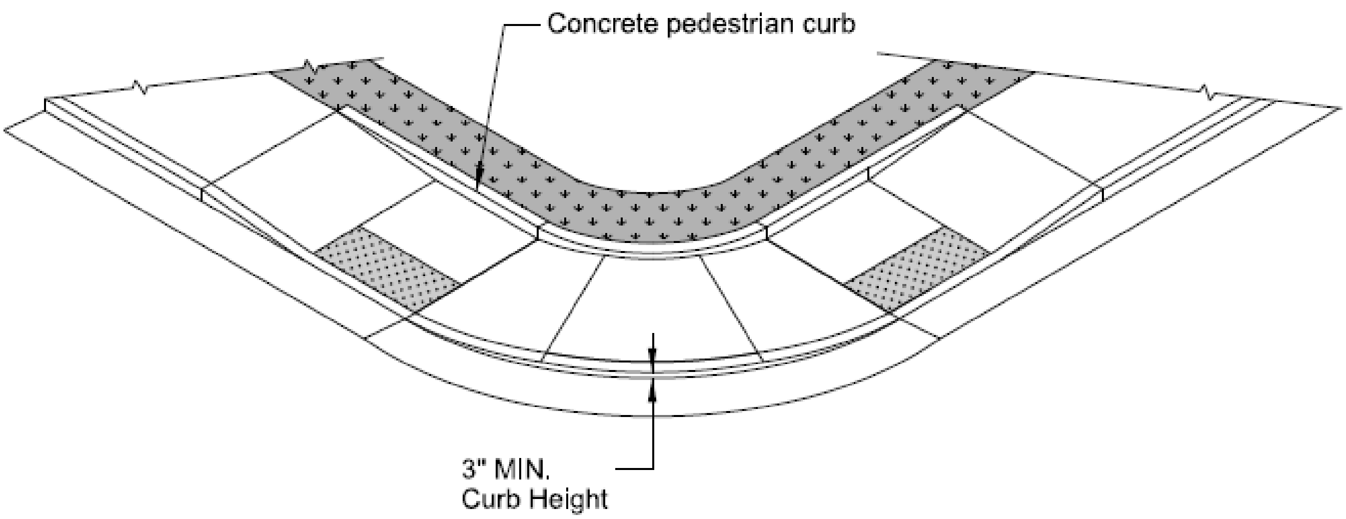


Figure 12-11 - Parallel Curb Ramp Example

In a blended transition the elevation of the sidewalk is slowly lowered to the street level at the corner. The maximum grade of a blended transition is 5.0% and the maximum cross slope is 2.0%. Blended transitions are typically seen in dense urban areas such as central business districts, around stadiums, or in main street environments. Blended transitions present an opportunity for turning vehicular traffic to traverse the sidewalk and thereby pose a safety risk to pedestrians. Blended transitions also create challenges for the visually impaired because they provide limited directionality. Blended transitions should be used sparingly, and only where appropriate.

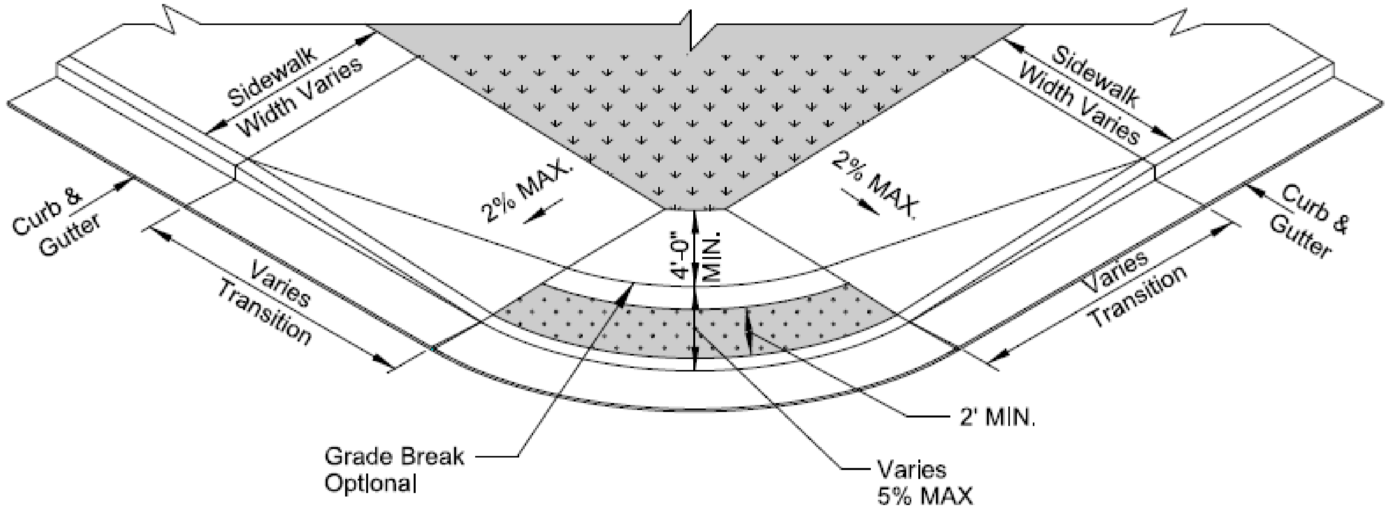


Figure 12-12 - Blended Transition/Depressed Corner Example

12.4 CURB RAMP TECHNICAL REQUIREMENTS

Although there are several types and configurations of curb ramps, including combinations of the types mentioned above, they all have the following general requirements:

Ramp Running Slope (R304.2.2) - Curb ramps shall have a maximum running slope of 8.33%. The running slope of a curb ramp is measured in the center of the ramp run in the direction of pedestrian travel. If the surrounding terrain requires a ramp to chase grade, the ramp is required to be no longer than 15 feet, regardless of the resulting slope**. Designers should target a running slope of 7.5% to provide for flexibility during construction.** Designing to the maximum allowable slopes does not leave any flexibility to those who must construct curb ramps in the field. The running slope of a turning space shall be 2.0% maximum.

Ramp Cross Slope (R304.5.3) - The maximum cross slope of a curb ramp shall be 2.0%. **Designers should target a cross slope of 1.5% to provide for flexibility during construction**. For street crossings that do not have stop or yield control, or at mid-block crossings, the cross slope is allowed to match the street or highway grade. See the PAR cross slope requirements for more information.

Ramp Width (R304.5.1) - The clear width of curb ramp runs, turning spaces, and blended transitions shall be 4 feet minimum. On CDOT projects, curb ramp runs, turning spaces, and blended transitions should be 5 feet in width. If the sidewalk the curb ramp is servicing is wider than 5 feet the ramp should match the width of the facility it is serving. Curb ramps that service shared-use paths should match the width of the path.

Grade Breaks (R304.5.2) - Grade breaks at the top and bottom of curb ramp runs shall be perpendicular to the ramp. Grade breaks are not allowed on the surface of the ramp run or turning space. Surfaces that meet at grade breaks shall be flush. When grade breaks are not perpendicular to the path of travel they pose challenges to users in wheelchairs because one wheel may strike the ramp before the others.

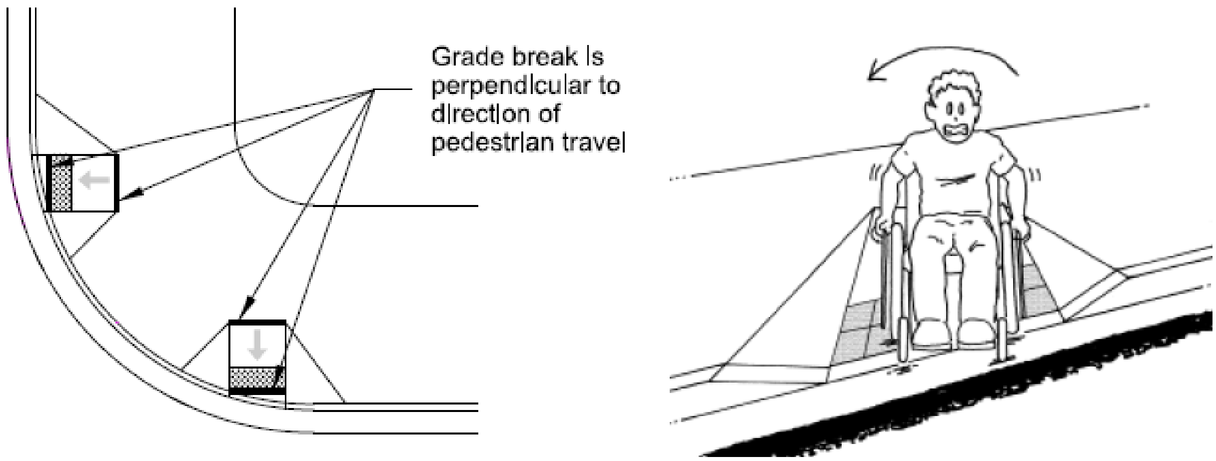


Figure 12-13 - Curb Ramp Grade Breaks

Turning Spaces (R304.2.1 & 304.3.1) - Turning spaces allow users to maneuver on and off the curb ramp and are required at the top or bottom of a curb ramp. Turning spaces are required at the top of a perpendicular curb ramp and at the bottom of a parallel curb ramp. The maximum running slope and cross slope of turning spaces shall be 2.0**%. Designers should target slopes of 1.5% to provide flexibility during construction**. At mid-block crossings or locations without yield or stop control, the cross slope of the turning space is allowed to equal the street or highway grade (R302.6). Turning spaces shall be 4 feet by 4 feet minimum. If the turning space is constrained by a vertical element on one or more sides, provide 5 feet in the direction of the street crossing.

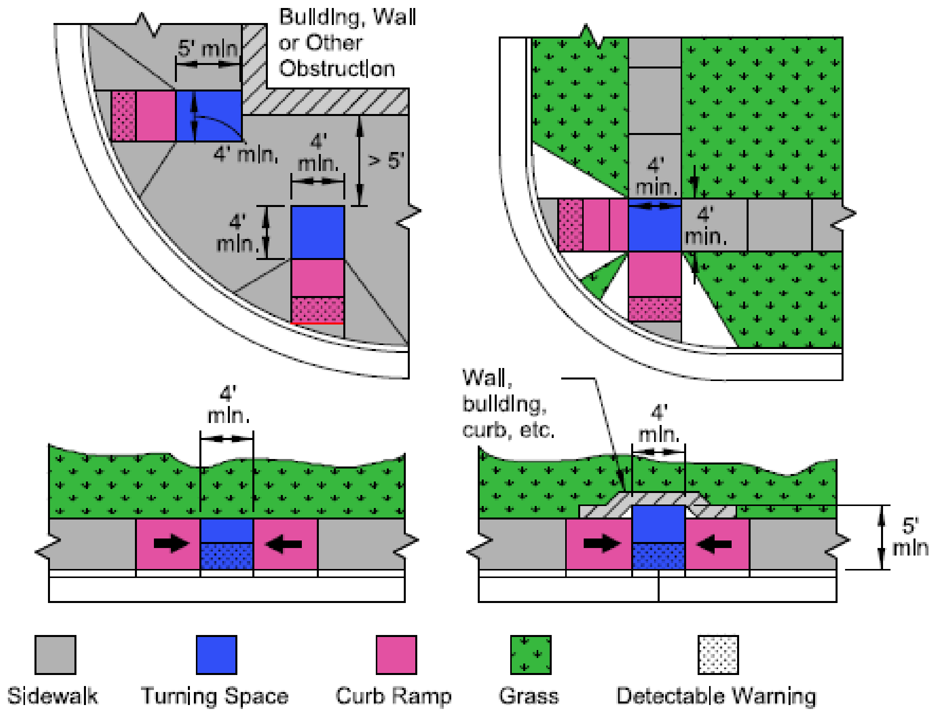


Figure 12-14 - Curb Ramp Turning Spaces

When the profile of the roadway being crossed has an excessive slope, the curb ramp cross slope should be transitioned slowly to the turning space. The transition shall be spread evenly over the length of the curb ramp.

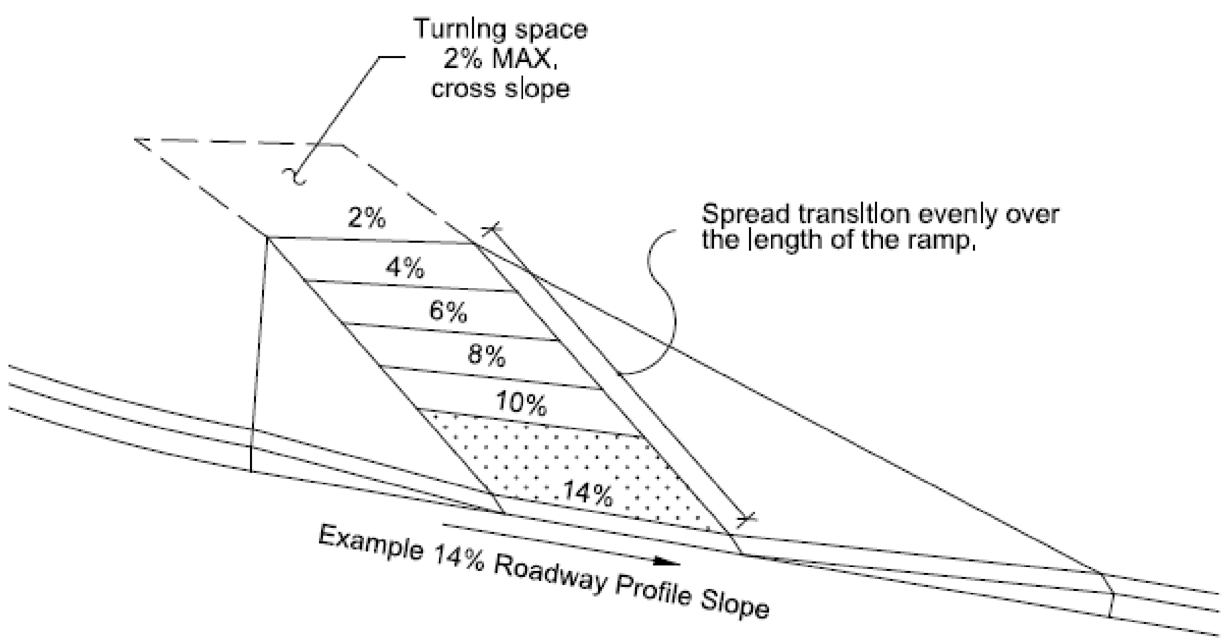


Figure 15 - Transitioning Steep Roadway Slopes

Clear Spaces (R304.5.5) - Beyond the bottom grade break of perpendicular ramps, a clear space 4-foot by 4-foot minimum shall be provided. This clear space must be within the pedestrian street crossing (crosswalk) and wholly outside of any vehicle travel lanes.

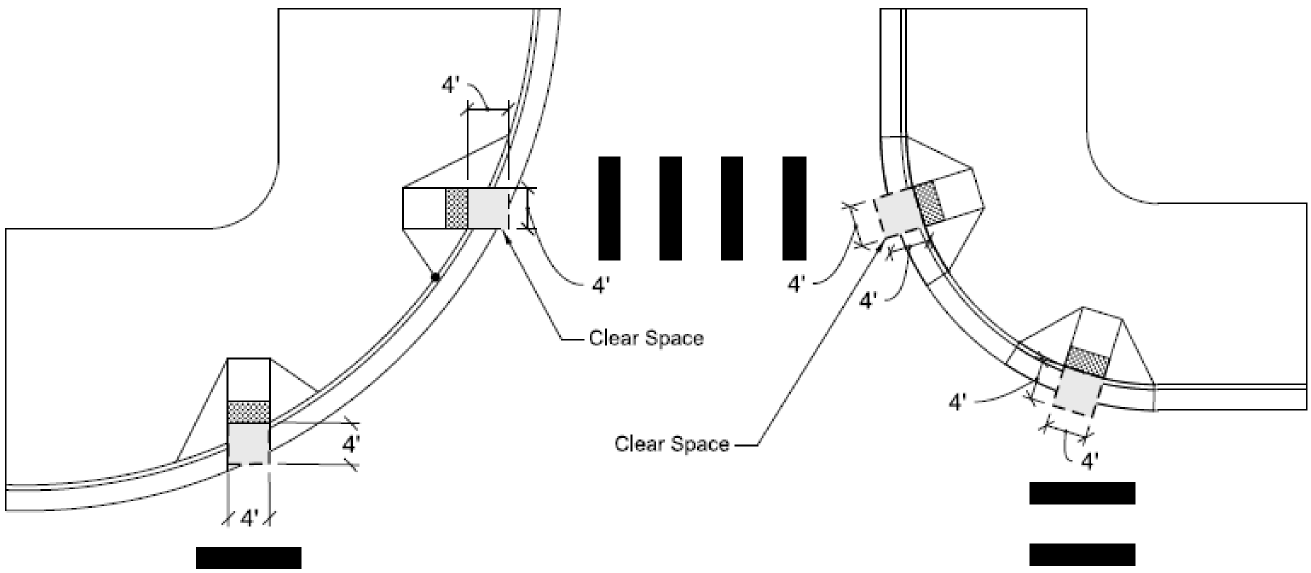


Figure 12-16 - Perpendicular Curb Ramp Clear Spaces

Flared Sides (R304.2.3) - Where a pedestrian can walk across a curb ramp, flared sides shall be provided to prevent a tripping hazard. Flared sides shall be sloped at 10.0% maximum and be measured parallel to the curb line. **A best practice is to design flares with slopes between 8.0% and 10.0%**, this helps clearly define the curb ramp from the sidewalk. Flared sides are only required where the curb ramp abuts a portion of a pedestrian circulation path. If access to a ramp flare is blocked from pedestrian travel by an item such as street furniture or a utility, then the flare slope may exceed 10%.

A vertical curb face may be used if the ramp abuts a non-walkable surface. Vertical curb faces can be beneficial to pedestrians with visual disabilities because they align pedestrians in the direction of the street crossing. Flared sides may be used when the ramp abuts a non-walkable surface, in this situation the allowable slope of the flare is at the designer's discretion. Using flared sides can be beneficial in protecting the ramp from vehicle strikes such as from snowplows, however, they do not provide the directionality benefits that vertical faces do.

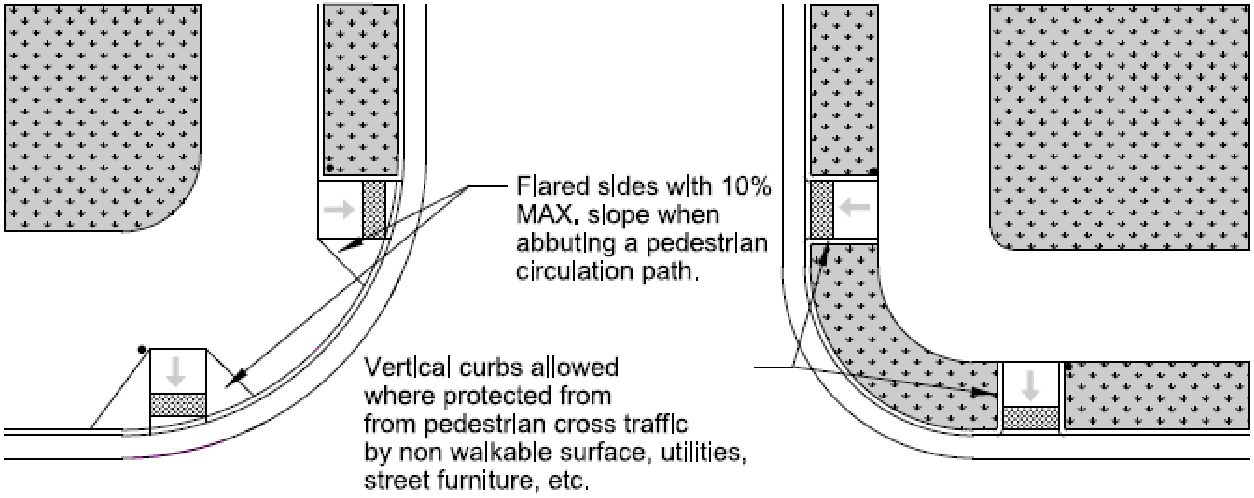


Figure 12-17 - Flared Sides

Counter Slope - The transition from the curb ramp to the roadway surface must be flush. The counter slope of gutter or street must be 5.0% or less. The algebraic difference of the ramp run and the street counter slope shall not exceed 13.33%. A rapid change in grade at the bottom of a ramp run can be difficult for wheelchair users to traverse. A best practice is to limit the algebraic difference between the ramp run and the counter slope to 11.0%, unless special accommodations are made. If the algebraic difference of the ramp and the counter slope exceeds 11.0%, a level strip should be provided to ease the transition from the ramp to the street.

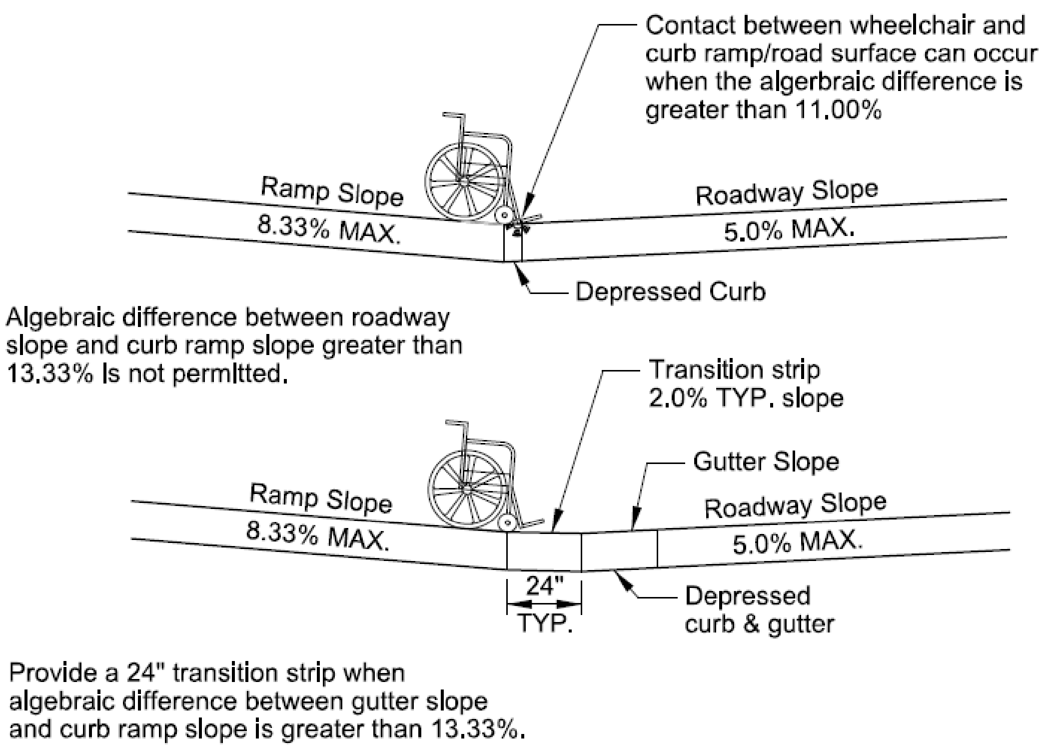


Figure 12-18 - Curb Ramp Counter Slope Requirements

12.5 DETECTABLE WARNING SURFACES

Detectable warning surfaces (DWS) provide an indication to individuals with disabilities that they are entering a potentially dangerous area. This is communicated through a distinct patterned surface consisting of truncated domes. DWS should be detectable by cane or underfoot and are intended to differentiate the boundary between the pedestrian realm and vehicular routes when a raised curb face is missing. DWS provide a cue to pedestrians with visual impairments and are required to contrast visually with the surrounding surface (light on dark or dark on light).

The truncated dome pattern of the DWS should be aligned so that the rows of domes are parallel to the direction of pedestrian travel. This alignment allows users in wheelchairs to more easily track between the domes and avoid excessive vibration, which can be uncomfortable to individuals with a spinal cord injury. The orientation of the truncated domes is not intended to provide wayfinding assistance (orient users in the direction of the crossing) for pedestrians with visual disabilities. While a best practice, this is not a requirement and cannot always be accomplished.

DWS shall extend 2 feet in the direction of pedestrian travel and are typically placed at the back of curb. DWS shall extend the full width of a curb ramp (excluding flared sides), blended transition, or turning space. DWS shall extend the full width of pedestrian rail crossings or shared-use path crossing. At boarding platforms for railways or buses, DWS shall be placed along the full length of the platform. At boarding areas at the sidewalk or street level the DWS shall extend the full length of the transit stop. When a border is required for installation, such as when pavers are used, the border shall be no more than 2 inches in width.

For the technical requirements of the truncated dome size and spacing, see the PROWAG section R305.1 or CDOT Standard Plan M-608-1.

Of the DWS allowed on CDOT projects, pavers require the most ongoing maintenance over time. This should be considered when specifying their usage on a project.

Placement - Detectable warning surfaces must be installed at all pedestrian street and rail crossings (R208.1). The PROWAG advises where DWS should be placed in Advisory R208.1. It recommends that DWS not be placed at residential driveway crossings or at locations where the pedestrian right-of-way continues across driveway aprons. However, it does recommend that DWS be placed at commercial driveways when yield or stop control is present. This guidance is not necessarily comprehensive and may be confusing to designers. For example, small commercial centers might have less driveway traffic than large apartment complexes but require DWS. Sound engineering judgement should be applied regarding the application of DWS at driveway crossings. Conditions that make a driveway function more like a street crossing may warrant the use of DWS, regardless of the land use category. Types of driveway locations that may warrant the use of DWS include:

- If there is the presence of a traffic control device that indicates the driveway functions more like a street crossing than a driveway (e.g. lane or pavement markings, signals, stop signs).
- Where a sidewalk descends to meet the grade of the street at the driveway crossing.
- Where the sidewalk changes material and ceases to be clearly defined as sidewalk, for example changing from concrete sidewalk to asphalt that looks more similar to the street at the driveway.

Parallel Curb Ramps - On parallel curb ramps the DWS shall be placed in the turning space at the back of curb where the curb face is missing.

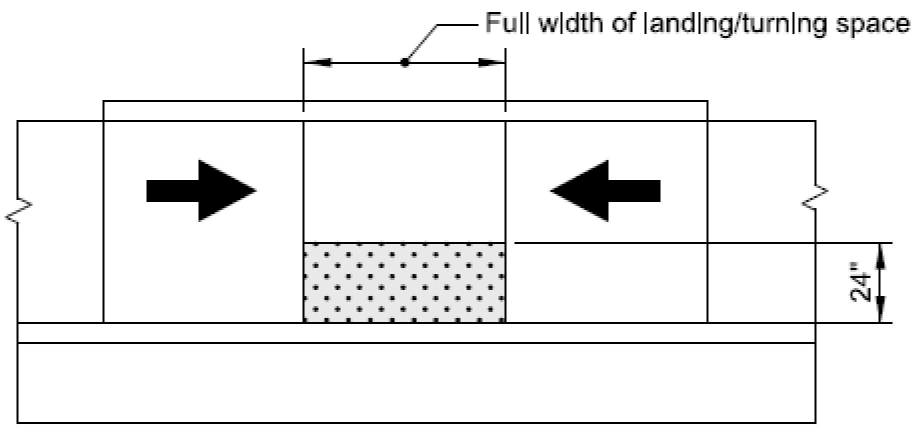


Figure 12-19 - Parallel Curb Ramp DWS Placement

Perpendicular Ramps - The placement of the DWS on perpendicular curb ramps varies depending on the location of the grade break at the bottom of the ramp. Generally, the DWS is placed at the back of curb where the curb face is missing. In situations where the ramp is located on a radius and the grade break at the bottom of the ramp is less than 5 feet from the back of curb, the DWS is placed at the bottom of the ramp (see Figure 20).

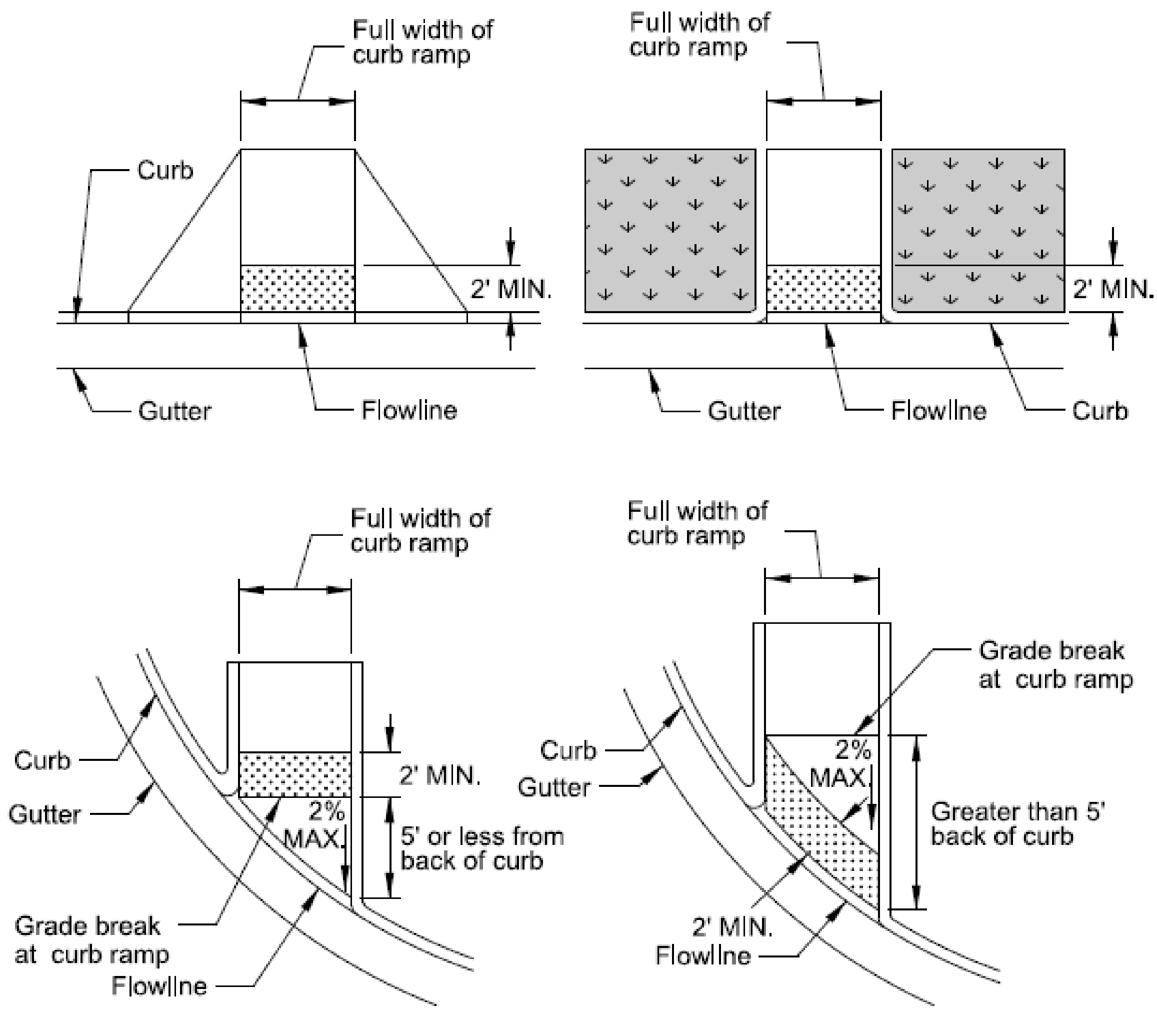


Figure 12-20 - Perpendicular Curb Ramp DWS Placement

Pedestrian Refuge Islands - When a pedestrian refuge island is 6 feet wide or greater from the face of curb to face of curb, DWS shall be installed. When installed the detectable warning surfaces must be separated by a surface without DWS that is a minimum of 2 feet in length (parallel to pedestrian travel). If this 2-foot space cannot be provided, DWS shall not be installed because the island is not wide enough to be considered a pedestrian refuge.

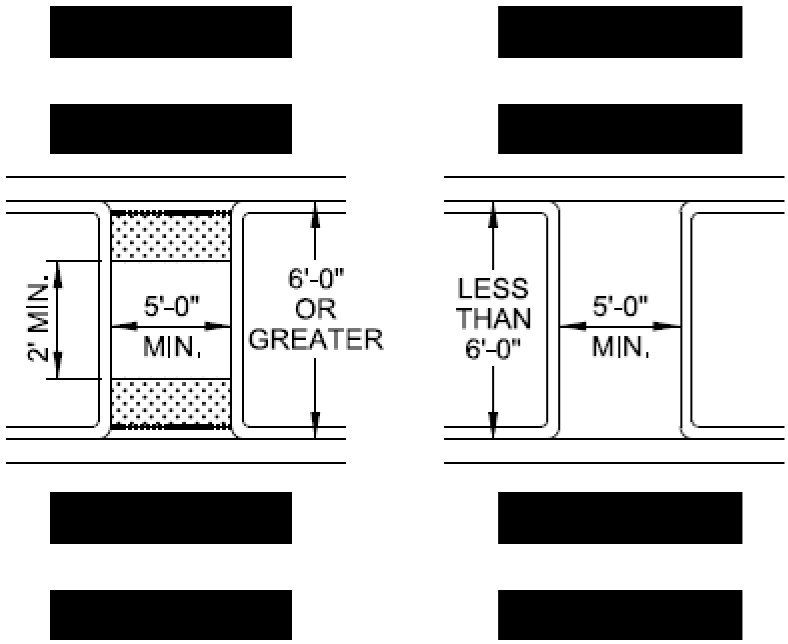


Figure 12-21 - Pedestrian Refuge DWS Placement

Railroad Crossings - When pedestrian rail crossings are not within the street or highway, detectable warning surfaces shall be placed on each side of the rail crossing. The DWS shall be a minimum of 6 feet from the nearest rail and a maximum of 15 feet from the nearest rail. If pedestrian crossing gates are present, the DWS shall be placed so that the pedestrian encounters the DWS before the crossing gate when traveling toward the rail crossing.

12.6 PEDESTRIAN CROSSING CONTROLS

When pedestrian activated crossing controls are provided they must meet the requirements set forth in the MUTCD, specifically sections 4E.08 through 4E.13. An accessible pedestrian signal (APS) is a device that communicates information to pedestrians about the street crossing through audible tones and vibrotactile surfaces. Operable parts on items such as pedestrian push buttons and APS shall comply with section R403 of the PROWAG.

Consistency throughout the pedestrian system is critical. On CDOT signal projects each location should be evaluated to determine if there is a need for APS through the application of CDOT's APS protocol.

https://www.codot.gov/business/civilrights/ada/assets/cdot_aps_protocol_march_2017.pdf

In addition to the requirements in the MUTCD and the PROWAG, the following should be taken into consideration when installing pedestrian crossing controls for pedestrian accessible routes:

- Pedestrian push buttons should be located as close as practical to the curb ramp they are servicing while at the same time permit operation from a clear and level space.

- The pedestrian push button location should not interfere with the use of the curb ramp or the sidewalk.
- If a push button cannot be practically located within the recommended area, consider moving it back and adjusting the pedestrian crossing time.
- Do not place the push button pole in the pedestrian access route.
- Pedestrian push buttons shall be mounted between 15 and 48 inches above the surface from which they are being accessed; the preferred mounting height is 42 inches.
- Pedestrians should not have to reach more than 10 inches to access a push button.
- A firm, stable, slip resistant surface must be provided to allow for a forward or parallel reach to the pedestrian push button from a wheelchair.
- Pedestrian push buttons should not be placed adjacent to the running slope of a curb ramp.
- Pedestrian push buttons should be located on the side of the pole from which the pedestrian accesses the button. Pedestrians should not have to reach around the pole to access the button.

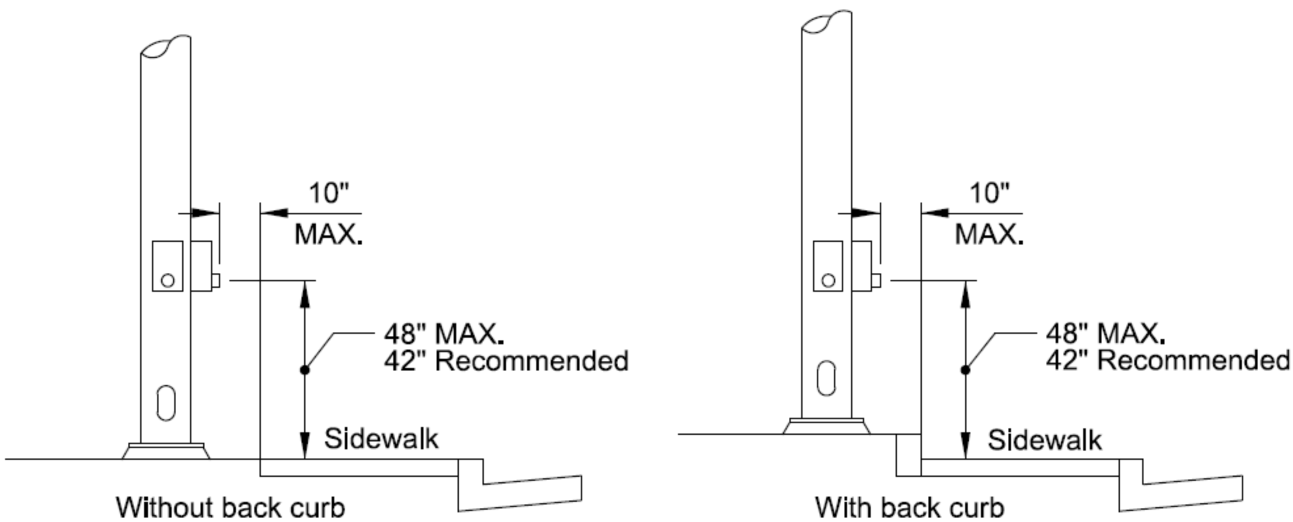


Figure 12-22 - Ped Push Button Reach Ranges

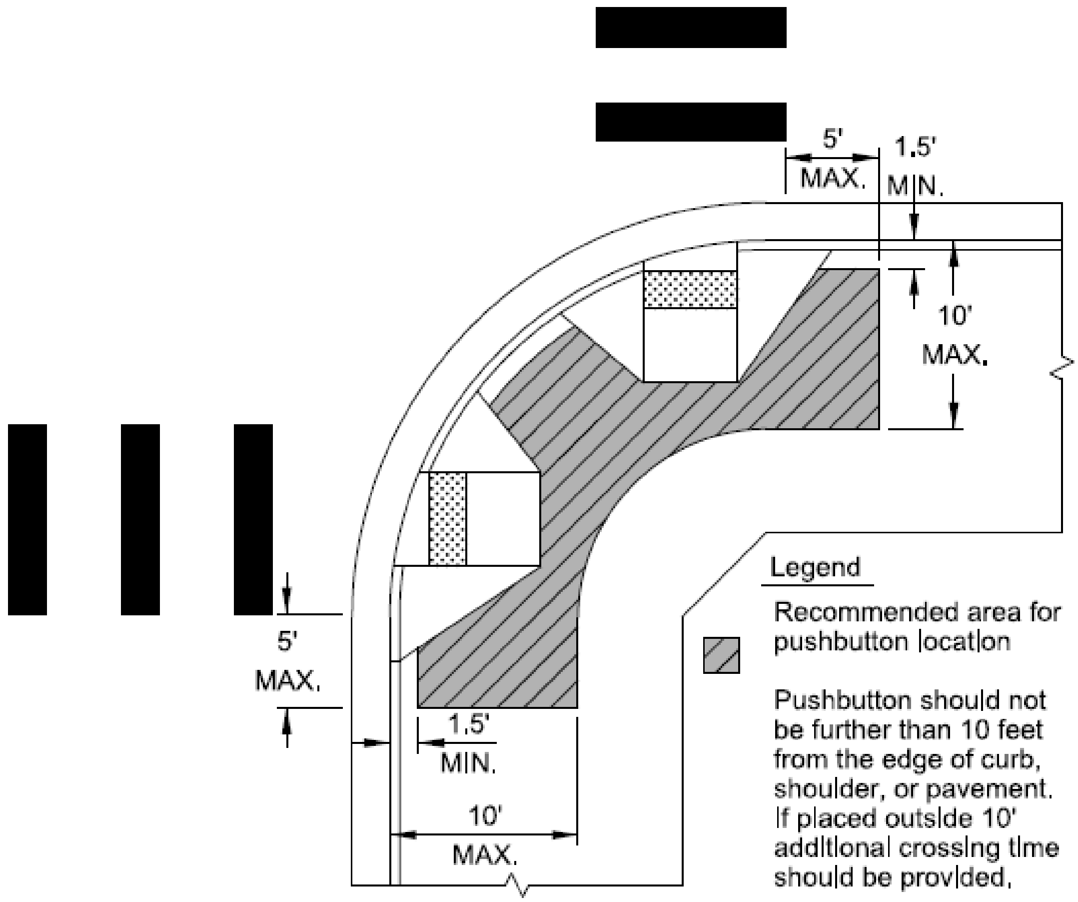


Figure 12-23 - Ped Push Button Placement

12.7 PEDESTRIAN RAMPS & LANDINGS

Pedestrian ramps that traverse changes of elevation should use a running slope between 5.0% and 8.33%. Where possible, the flattest running slope should be used to accommodate the widest possible range of users. Wheelchair users with disabilities affecting their arms, or individuals with low stamina, can have difficulties using pedestrian ramps. For this reason level landing areas are provided at regular intervals on pedestrian ramps. The vertical rise of any ramp run shall not exceed 30 inches without a level landing space. Ramps shall have level landing spaces at both the top and the bottom of each ramp run. Landing spaces should be 5 feet in length and match the width of the ramp. If the landing space is located at a 90-degree turn in the ramp it should have a minimum dimension of 5 feet by 5 feet.

Any ramp with a vertical rise greater than 6 inches shall provide handrails. The requirements for pedestrian handrails are described in detail in Section R409 of the PROWAG.

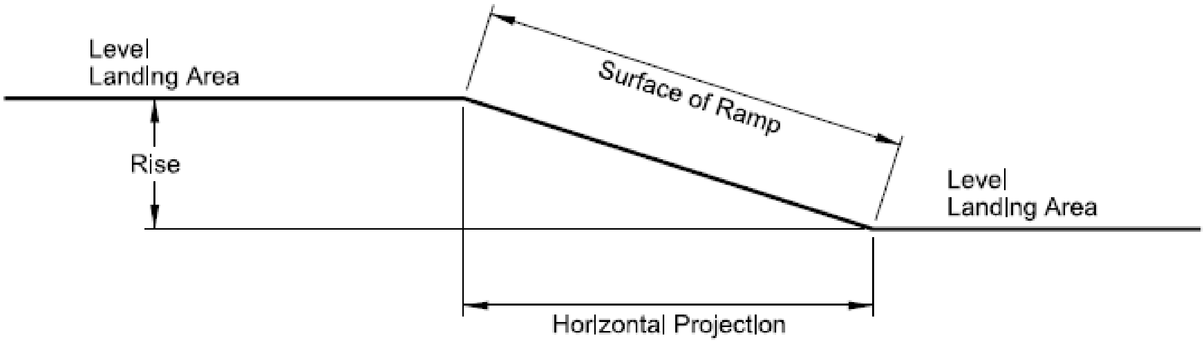


Figure 12-24 - Pedestrian Ramp Elements

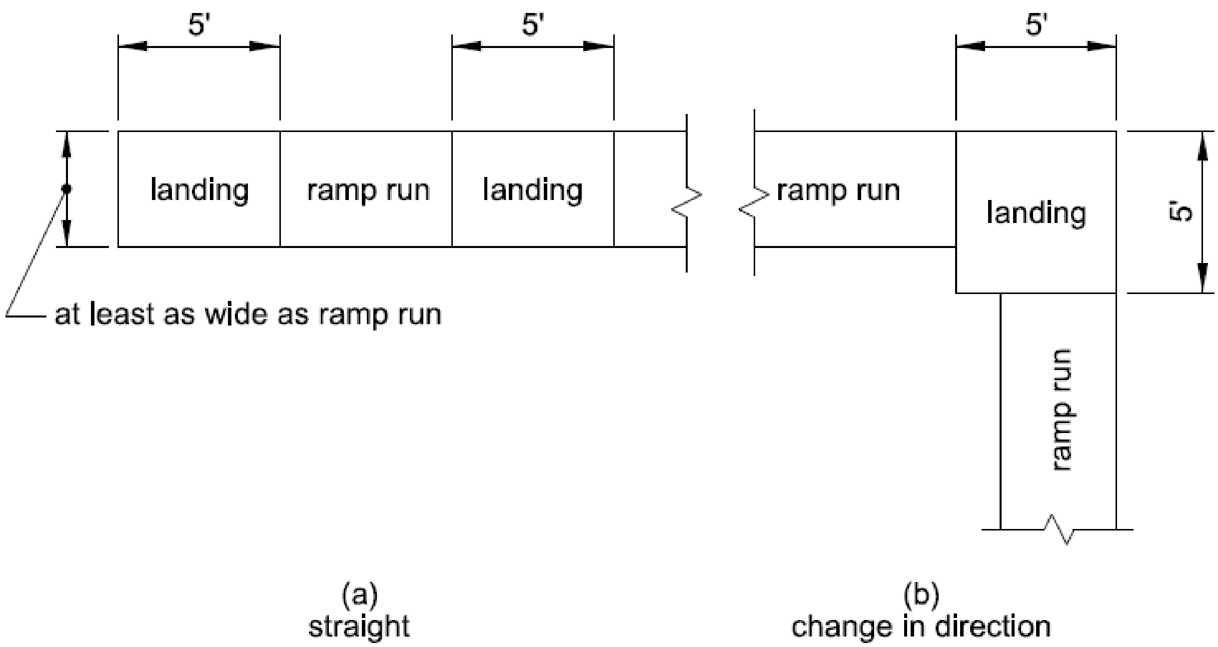


Figure 12-25 - Pedestrian Ramp Turning Space Requirements

CHAPTER 13
ALTERNATE STANDARDS (LOW VOLUME ROADS)

13.0 INTRODUCTION

Design guidelines for very low5-volume roads may differ from those of higher volume roads. AASHTO's *Geometric Design Guidelines for Very Low-Volume Roads (ADT ≤400)* **(1)** defines the needs of these roadways and the criteria to meet those needs. When defined as a low-volume roadway, this design guideline may be used in place of guidelines defined in the Green Book, *A Policy on Geometric Design of Highways and Streets (PGDHS)* **(2),** if applicable.

13.1 DEFINITION AND CHARACTERISTICS

A very low-volume local road has a functional classification of local road, and features a design average daily traffic volume of 400 vehicles per day, at most. Functionally classified collectors may also follow these guidelines so long as the design average daily traffic volume does not exceed 400 vehicles per day. These low volumes significantly reduce the opportunities for accidents to occur. Low volume roads also cater to local traffic familiar with the roadway; local drivers typically know and can anticipate design abnormalities. Design guidelines for very low-volumes roadways may be less strict than for roadways with higher volumes or less familiar drivers.

13.2 LOW-VOLUME FUNCTIONAL CLASSIFICATIONS

Very low-volume roads are divided into six rural functional classifications and three urban functional classifications. They are as follows:

Rural Roads

- Rural Major Access Roads
- Rural minor Access Roads
- Rural Industrial/Commercial Access Roads
- Rural Agricultural Access Roads
- Rural Recreational and Scenic Roads
- Rural Resource Recovery Roads

Urban Roads

- Urban Major Access Streets
- Urban Residential Streets
- Urban Industrial/Commercial Access Streets

13.2.1 Rural Major Access Roads

Rural major access roads are defined by the following characteristics:

- Provide through or connecting service between other local roads or higher type facilities.
- They have significant local continuity and may operate at relatively high speeds.
- Due to through traffic, some traffic may include unfamiliar drivers.

- Roads are usually paved.

Collector roads that meet the definition of a very low-volume local road should be classified as a rural major access road.

13.2.2 Rural Minor Access Roads

Rural minor access roads are defined by the following characteristics:

- Serve almost exclusively to provide access to adjacent property.
- Such roads are used predominantly by familiar drivers.
- Speeds are generally low for the local environments.
- Roads are frequently narrow and sometimes may function as one-lane roads.
- Roads can be either paved or unpaved.
- Traffic is primarily composed of passenger vehicles
- Roads need to be accessible to school buses, fire trucks, etc.

13.2.3 Rural Industrial/Commercial Access Roads

Rural industrial/commercial access roads are defined by the following characteristics:

- May generate a significant proportion of truck or other heavy vehicle traffic
- Generally, provide access from commercial land use to the regional highway network.
- Roads are typically very short and do not serve any through traffic.
- Roads may be either paved or unpaved.

13.2.4 Rural Agricultural Access Roads

Rural agricultural access roads are defined by the following characteristics:

- Primarily provide access to fields and farming operations.
- Vehicle types included slow-moving vehicles such as farm equipment
- Drivers typically consist of repeat users who are familiar with the roadway characteristics.
- Roads are often unpaved

13.2.5 Rural Recreational and Scenic Roads

Rural recreational and scenic roads are defined by the following characteristics:

- Serve specialized land uses, such as parks, tourist attractions, campsites, etc.
- Traffic consists primarily of unfamiliar drivers.
- Traffic consists of low volumes of truck traffic
- Roads may carry highly seasonal traffic volumes.
- May accommodate a wide range in speeds and trip lengths.
- Roads can be either paved or unpaved.

13.2.6 Rural Resource Recovery Roads

Rural resource recovery roads are defined by the following characteristics:

- Serve logging or mining operations.
- Typically found only in rural areas.
- Drivers are typically professional drivers with large vehicles.
- Traffic operations are typically enhanced with radio communication between drivers.
- Most roads are unpaved.

13.2.7 Urban Major Access Streets

Urban major access streets are defined by the following criteria:

- Provide access to adjacent property and through traffic to other local roads.
- Generally short but serve slightly more traffic than most local roads.

Collector roads that meet the definition of a very low-volume local road should be classified as an urban major access street.

13.2.8 Urban Residential Streets

Urban residential streets are defined by the following characteristics:

- Typically serve to provide access to single and multiple family residences in urban areas.
- Drivers generally include only residents and their visitors.
- Large trucks are rare.
- Provide accessibility for fire trucks and school buses

13.2.9 Urban Industrial/Commercial Access Streets

Urban industrial/commercial access streets are defined by the following characteristics:

- May generate a substantial volume of trucks or other heavy vehicles.
- Generally, provide access from commercial land use to the regional highway network.
- Roads are typically very short and may not carry traffic from smaller streets.
- Roads may be either paved or unpaved.

If a roadway definition meets more than one functional classification, the stricter guidelines shall be applied.

13.3 LOW VOLUME DESIGN APPLICATIONS

The design guidelines defined in the *Geometric Design Guidelines for Very Low-Volume Roads* **(1)** provide less strict design criteria however they do not compromise safety when applied to very low-volume roadways with familiar drivers. The purpose of the low volume guideline is to provide a recommended range of values, and not to be a replacement of detailed design manuals. These guidelines allow for flexibility in designs to accommodate specific needs.

13.3.1 Design and Operation Speed

The design guidelines presented are a function of speed, as follows:

- Low speed – 0 to 45 mph
- High speed – < 45 mph

13.3.2 Traffic Volumes

Traffic volumes on very low-volume roads are stratified into three levels for purposes of these design guidelines. The volume ranges are:

- 100 vehicles per day or less
- 100 to 250 vehicles per day
- 250 to 400 vehicles per day

13.4 CROSS SECTION DESIGN

Cross section design criteria for lower volume roads generally address total roadway width rather than having separate criteria for lane and shoulder width.

13.4.1 Very Low-Volume Local Roads in Rural Areas Cross Section

Table 13-1 illustrates the total roadway width for the six low volume functional classifications for rural conditions. These cross section widths are based on the expected user vehicles.

Total Roadway Width (ft) by Functional Classification						
Design Speed (mph)	**Major Access**	**Minor Access**	**Recreational and Scenic**	**Industrial/ Commercial Access**	**Resource Recovery**	**Agricultural Access**
15	-	18.0	18.0	20.0	20.0	22.0
20	-	18.0	18.0	20.0	20.0	24.0
25	18.0	18.0	18.0	21.0	21.0	24.0
30	18.0	18.0	18.0	22.5	22.5	24.0
35	18.0	18.0	18.0	22.5	22.5	24.0
40	18.0	18.0	20.0	22.5	-	24.0
45	20.0	20.0	20.0	23.0	-	26.0
50	20.0	20.0	20.0	24.5	-	-
55	22.0	-	22.0	-	-	-
60	22.0	-	-	-	-	-
Note: Total Roadway width includes the width of both traveled way and shoulders.						

Table 13-1 (Exhibit 1 of the *Geometric Design of Very Low-Volume Local Roads* (1))
Total Roadway Widths for Rural Conditions

13.4.2 Very Low-Volume Local Roads in Urban Areas Cross Section

Table 13-2 illustrates the total roadway width for the urban conditions based on development density.

Development Density	Total Roadway Width (ft)
Low	20 to 28
Medium	28 to 34
Note: Low density represents 2.0 or fewer dwelling units per acre; medium development density represents 2.1 to 6.0 dwelling units per acre.	

Table 13-2 (Exhibit 2 of the *Geometric Design of Very Low-Volume Local Roads* (1)) Total Roadway Widths for Urban Conditions

The lower end of the total roadway width range is intended for streets with mostly off-street parking such as driveways, typically in a subdivision setting. The upper end of the total roadway width range is intended for streets with recurrent parking on one side of the street. For streets with parking frequently occurring on both sides of the street, total roadway widths that exceed what is shown in Table 13-2 may be used.

13.5 HORIZONTAL DESIGN

13.5.1 Horizontal Curve Design

Horizontal roadway design is commonly illustrated as a relationship between design speed and roadway alignments. Curves are a function of speed, alignment, superelevation, and side friction. A key parameter that represents the friction demand for a vehicle traversing a horizontal curve is the side friction factor, which can be estimated using Equation 13-1.

$$\boldsymbol{f = \frac{V^2}{15R} - 0.01e} \qquad [13\text{-}1]$$

where,

f = side friction factor
V = vehicle speed (mph)
R = radius of curve (feet)
e = rate of roadway superelevation (percent)

A fundamental objective in horizontal curve design is to select a radius of curve, R, such that the side friction factor, *f*, of a vehicle traversing the curve at the design speed does not exceed a specified threshold value. To achieve this, Equation 13-2 can be used.

$$Rmin = \frac{V^2}{15(0.01emax+fmax)} \quad [13\text{-}2]$$

where,

R_{min}= minimum curve radius (feet)

e_{max} = maximum rate of superelevation permitted

f_{max} = *maximum side friction factor*

Minimum curve radii for streets with higher volumes can be found in the *PGDHS* **(2)** and are shown in Table 13-3.

Design Speed (mph)	Maximum Design Side Friction Factor (f_{max})	Minimum Radius (ft), R_{min}				
		Maximum Superelevation, e_{max}				
		4%	6%	8%	10%	12%
15	0.320	42	39	38	36	34
20	0.270	86	81	76	72	68
25	0.230	154	144	134	126	119
30	0.200	250	231	214	200	188
35	0.180	371	340	314	292	272
40	0.160	533	485	444	410	381
45	0.150	711	643	587	540	500
50	0.140	926	833	758	694	641
55	0.130	1190	1060	960	877	807
60	0.120	1500	1330	1200	1090	1000

Table 13-3 [Developed from Table 3-7 of the *Geometric Design of Very Low-Volume Local Roads* (1)] Minimum Radius Using Limiting Values of *e* and *f*

Low speed urban streets are those urban roadways with design speeds of less than 45 mph. Superelevation rates greater than 6% are not recommended for such streets because higher rates would be inappropriate for low-speed conditions. Table 13-3 also illustrates the minimum curve radii for streets with higher volumes, but low speed (45 mph or less) in an urban setting.

The minimum radii for new construction of low volume roadways is provided in the *Geometric Design Guidelines for Very Low-Volume Roads* **(1).** Design guidelines are further refined for each category. See *Very-Low Volume Roads* **(1)** for design speed, recommended reduced design speed, and corresponding minimum radii as a function of maximum superelevation for:

- **Rural Major Access, Minor Access, and Recreations and Scenic Roads (250 Vehicles per Day or Less)**. See Exhibit 5 in the *Geometric Design Guidelines for Very Low-Volume Roads* **(1)**

- **Rural Major Access, Minor Access, and Recreational and Scenic Roads (250 to 400 Vehicles per Day)**. See Exhibit 6 in the *Geometric Design Guidelines for Very Low-Volume Roads* **(1)**
- **Rural Industrial/Commercial Access, Agricultural Access, and Resource Recovery Roads**. See Exhibit 7 in the *Geometric Design Guidelines for Very Low-Volume Roads* **(1)**
- **Urban Major Access Streets (250 Vehicles per Day or Less) and Urban Residential Streets**.
- **Urban Major Access Streets (250 to 400 Vehicles per Day)**.
- **Urban Industrial/Commercial Access Streets**.

13.5.1.1 Existing Roadways

The existing horizontal curve geometry should be considered acceptable for roadways with design speeds exceeding 45 mph with the nominal speed being within 10 mph of the operating speed, and so long as there is no documented safety concern. The existing horizontal curve geometry should be considered acceptable for roadways with design speeds equal to or less than 45 mph with the nominal speed being within 20 mph of the operating speed, and so long as there is no documented safety concern.

13.5.2 Superelevation and Superelevation Transitions

Superelevation and superelevation transitions shall follow criteria set forth in the *PGDHS* **(2)**. When using design criteria as discussed in Section 13.5.1, Horizontal Curve Design, use the reduced design speed when determining superelevation and superelevation transitions.

13.5.3 Stopping Sight Distance

Stopping sight distance for low-volume roadways differs from higher volume roadways where a vehicle must come to a complete stop, in that the driver may avoid an obstruction instead of needing to stop before it. The same stopping sight distance equation is used with alternative variables.

Low-volume roadways are categorized into two risk categories; lower risk and higher risk. “Lower risk” locations are locations away from intersections, narrow bridges, at grade railroad/highway crossings, sharp curves, and steep grades. “Higher risk” locations include intersections, narrow bridges, at grade railroad/highway crossings, sharp curves, and steep grades. Table 13-4 illustrates the design sight distance guidelines for new construction of very low-volume local roads.

Minimum Sight Distance (ft)				
Design Speed (mph)	0-100 veh/day	100-250 veh/day		250-400 veh/day
	All Locations	All Locations	Lower Risk Locations	Higher Risk Locations
15	65	65	65	65
20	90	90	95	95
25	115	115	125	125
30	135	135	165	165
35	170	170	205	205
40	215	215	250	250
45	260	260	300	300
50	310	310	350	350
55	365	365	405	405
60	435	435	470	470

Table 13-4 (Exhibit 8 of the *Geometric Design of Very Low-Volume Local Roads* (1))
Design Sight Distance Guidelines for New Construction of Very Low-Volume Local Roads

13.5.3.1 Sight Distance on Horizontal Curves

Similar to higher volume roadways, stopping sight distance on a horizontal curve is represented as a chord of a radius. Table 13-5 illustrates the minimum values of the middle ordinate for lower and higher risk low-volume roadways. The width on the inside of the curve is measured from the centerline of the inside lane. “Lower risk” locations are locations away from intersections, narrow bridges, at grade railroad/highway crossings, sharp curves, and steep grades. “Higher risk” locations include intersections, narrow bridges, at grade railroad/highway crossings, sharp curves, and steep grades.

Design Speed (mph)	Stopping Sight Distance (ft)	Width on Inside of Curve Clear of Sight Obstructions (ft)								
		Lower Risk (All locations 0-100 vpd and "low risk" locations for 100-250 vpd)								
		Radius of Curvature (ft)								
		50	100	200	500	1000	2000	5000	10000	20000
15	65	10.2	5.2	2.6	1.1	0.5	0.3	0.1	0.1	0.0
20	90	--	10.0	5.0	2.0	1.0	0.5	0.2	0.1	0.1
25	115	--	--	8.2	3.3	1.7	0.8	0.3	0.2	0.1
30	135	--	--	11.3	4.5	2.3	1.1	0.5	0.2	0.1
35	170	--	--	--	7.2	3.6	1.8	0.7	0.4	0.2
40	215	--	--	--	11.5	5.8	2.9	1.2	0.6	0.3
45	260	--	--	--	16.8	8.4	4.2	1.7	0.8	0.4
50	310	--	--	--	--	12.0	6.0	2.4	1.2	0.6
55	365	--	--	--	--	16.6	8.3	3.3	1.7	0.8
60	435	--	--	--	--	23.6	11.8	4.7	2.4	1.2
		Higher Risk ("high risk locations for 250-400 vpd and all locations 250-400 vpd)								
15	65	10.2	5.2	2.6	1.1	0.5	0.3	0.1	0.1	0.0
20	95	--	11.1	5.6	2.3	1.1	0.6	0.2	0.1	0.1
25	125	--	--	9.7	3.9	2.0	1.0	0.4	0.2	0.1
30	165	--	--	16.8	6.8	3.4	1.7	0.7	0.3	0.2
35	205	--	--	--	10.5	5.2	2.6	1.1	0.5	0.3
40	250	--	--	--	15.5	7.8	3.9	1.6	0.8	0.4
45	300	--	--	--	22.3	11.2	5.6	2.3	1.1	0.6
50	350	--	--	--	--	15.3	7.7	3.1	1.5	0.8
55	405	--	--	--	--	20.4	10.2	4.1	2.1	1.0
60	470	--	--	--	--		13.8	5.5	2.8	1.4

Table 13-5 (Exhibit 10 of the *Geometric Design of Very Low-Volume Local Roads* (1))
Design Guidelines for Sight Distance on Horizontal Curves for New Construction of Very Low-Volume Local Roads

Table 13-6 illustrates the rate of vertical curvature, K, for crest vertical curves on very low-volume roadways. Sag vertical curves should default to values identified in the *PGDHS* **(2)**.

Design Speed (mph)	Stopping Sight Distance (ft)	Rate of Vertical Curvature, K	
		Calculated	Design
		Lower Risk	
15	65	2.0	2
20	90	3.8	4
25	115	6.1	7
30	135	8.4	9
35	170	13.4	14
40	215	21.4	22
45	260	31.3	32
50	310	44.5	45
55	365	61.7	62
60	435	87.7	88
		Higher Risk	
15	65	2.0	2
20	95	4.2	5
25	125	7.2	8
30	165	12.6	13
35	205	19.5	20
40	250	29.0	29
45	300	41.7	42
50	350	56.8	57
55	405	76.0	76
60	470	102.4	103

Table 13-6 (Exhibit 12 of the *Geometric Design of Very Low-Volume Local Roads* (1))
Guidelines for Minimum Rate of Vertical Curvature to Provide Design Stopping Sight Distance on Crest Vertical Curves for New Construction of Very Low-Volume Local Roads

For additional information on vertical curve design, see the *PGDHS* **(2)**.

13.5.4 Intersection Sight Distance

Design guidelines for intersection sight distance for very low-volume roadways are only applicable for intersections where all roadways have less than 400 vehicles per day. Three types of intersections have been identified for clear sight triangle analysis:

- Intersections with no control (Case A)
- Intersections with stop control on the minor road (Case B)

- Intersections with yield control on the minor road (Case C)

13.5.4.1 Intersections with No Control (Case A)

Drivers of vehicles approaching an intersection with no control have been observed to decelerate regardless if views are obstructed or not, and regardless if a potential conflict is present or not. Drivers typically reduced their speed to approximately half of their running speed. With this in consideration, Table 13-7 illustrates the desired sight distance for each approach to an uncontrolled intersection. Table 13-8 illustrates adjustment factors to Table 13-7 values at intersections where an approach exceeds a 3% grade.

Design Speed (mph)	Sight Distance (ft)
15	70
20	90
25	115
30	140
35	165
40	195
45	220
50	245
55	285
60	325

Table 13-7 (Exhibit 14 of the *Geometric Design of Very Low-Volume Local Roads* (1)) Recommended Sight Distance Guidelines for New Construction of Intersections with No Traffic Control

Approach Grade (%)	Design Speed (mph)									
	15	20	25	30	35	40	45	50	55	60
-6	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.2	1.2	1.2
-5	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
-4	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1
-3 to +3	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
+4	1.0	1.0	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9
+5	1.0	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9
+6	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9

Table 13-8 (Exhibit 15 of the *Geometric Design of Very Low-Volume Local Roads* (1)) Adjustment Factors for Sight Distance Based on Approach Grade

13.5.4.2 Intersections with Stop Control on the Minor Road (Case B)

Approach sight triangles are not needed to minor roads approaching a stop control. This movement does require a departure sight triangle. The entire sight distance defined in the *PGDHS* **(2)** should be accommodated along the major roadway; however, in constrained scenarios sight distances identified in Table 13-3 should be considered a minimum. The vertex of the triangle should be located 14.4 ft from the edge of the traveled way.

13.5.4.3 Intersections with Yield Control on the Minor Road (Case C)

The entire sight distance defined in the *PGDHS* **(2)** should be accommodated along the major roadway; however, in constrained scenarios sight distances identified in Table 13-3 should be considered a minimum. Departure sight triangles are not needed for intersections with yield control on the minor road. The minor road approach does require clear sight triangles.

13.5.5 Roadside Design

Clear zone and traffic barrier warrants are two main elements of roadside design. The information provided below provides guidelines that may be used in lieu of, or to supplement, the policies from the AASHTO *Roadside Design Guide* **(3)** and the *PGDHS* **(2)**.

13.5.5.1 Clear Zone Width

Roadside clear zones applied to low volume roadways per the AASHTO *Roadside Design Guide* **(3)** have shown to provide only limited safety benefits and are not cost effective; however, clear zones should be accommodated when practical. Clear zone guidelines for very low-volume roads are as follow:

- In areas where a 6-foot shoulder can be provided with minimal costs, and minimal social and environment impacts.
- In areas where a 6-foot shoulder cannot be provided at a reasonable cost, or with considerable social or environmental impacts, a shoulder of less than 6 feet should may be used including designs with no clear recovery areas.
- Clear zone improvements should be considered in locations of higher risk for accidents.
- Clear zone improvements should be considered for special circumstances such as areas with higher heavy vehicle traffic, crash history, or future growth.

Clear zone design is flexible where unique project characteristics should be considered.

13.5.5.2 Traffic Barrier

Traffic barrier should be considered at the discretion of the engineer. Generally, traffic barrier is not cost effective or practical for very low-volume roadways.

REFERENCES

1. AASHTO. *Guidelines for Geometric Design of Very Low-Volume Local Roads (ADT ≤ 400),* American Association of State Highway Transportation Officials, Washington, D.C.: 2001

2. AASHTO. *A Policy on Geometric Design of Highways and Streets*, American Association of State Highway and Transportation Officials, Washington, D.C.: 2011.

3. AASHTO. *Roadside Design Guide.* American Association of State Highway and Transportation Officials, Washington, D.C.: 2011.

CHAPTER 14

BICYCLE AND PEDESTRIAN FACILITIES

14.0 INTRODUCTION

Multimodal transportation is a key element of CDOT's mission in providing improvements to the statewide transportation system. CDOT has adopted a Policy Directive and a Procedural Directive to improve the accommodation of bicycles and pedestrians in CDOT programs. Additionally, federal surface transportation law places a strong emphasis on creating a seamless transportation system that persons of all ages and abilities can utilize for safe and convenient access to jobs, services, schools and recreation.

The design requirements set forth in this chapter apply to all new construction and reconstruction projects. Although optional, they will also be considered for other projects when funding is available and where appropriate as determined by the Project Manager. Pursuant to Chief Engineer Policy Memo 7, *"it is imperative that surface treatment dollars are optimized in regards to maintaining the pavement surface. In that light, surface treatment dollars are not to be used to fund enhancements or other project related costs."*

The designer should also adhere to the requirements of CDOT Policy Directive 548.0 (Safety Considerations on 3R Projects) when considering improvements for bicycles and pedestrians on resurfacing, restoration, and rehabilitation projects. When bike and pedestrian facilities are warranted or requested, project managers will investigate other funding sources to supplement the primary funding for the project. If funds are not available, the Project Manager will document with a letter to the design file. The letter will specifically state what efforts were made to obtain other funding. Additionally, the project manager should determine if other sidewalk or bike path projects are planned in the same area to determine if there are opportunities to consolidate the projects.

14.0.1 Intent of Chapter 14 - Design of Bicycle and Pedestrian Facilities

This chapter provides detailed design criteria, standards, and guidance for the development of bicycle and pedestrian facilities. The material in this chapter is derived from the AASHTO *Policy on the Geometric Design of Streets and Highways* (*PGDSH*) **(1)**, the AASHTO *Guide for the Development of Bicycle Facilities* **(2)**, the AASHTO *Guide for the Planning, Design, and Operation of Pedestrian Facilities* **(3)**, the *Manual on Uniform Traffic Control Devices (MUTCD)* **(4)** and other federal documents or research as noted throughout this chapter. It is the intent of this chapter to be consistent with all of the criteria provided in federal or CDOT standards. This chapter is intended to provide those standards in one location and provide additional guidance (if possible) where none exists in the current standards or guidance documents.

14.0.2 CDOT Bike and Pedestrian Policy Directive 1602.0

In October of 2009, the Colorado Transportation Commission adopted CDOT's bicycle and pedestrian Policy Directive 1602.0. The purpose of this policy is

> *… to promote transportation mode choice by enhancing safety and mobility for bicyclists and pedestrians on or along the state highway system by defining the policies related to education and enforcement, planning, programming, design, construction, operation and maintenance of bicycle and pedestrian facilities and their usage.*

The intent of this policy is to:

> *It is the policy of the Colorado Transportation Commission to provide transportation infrastructure that accommodates bicycle and pedestrian use of the highways in a manner that is safe and reliable for all highway users. The needs of bicyclists and pedestrians shall be included in the planning, design, and operation of transportation facilities, as a matter of routine. A decision to not accommodate them shall be documented based on the exemption criteria in the procedural directive.*

14.0.3 CDOT Bike and Pedestrian Procedural Directive 1602.1

CDOT Procedural Directive 1602.1 requires the incorporation of bicycle and pedestrian considerations throughout CDOT's planning, programming, design, construction and maintenance operations (as well as educational and enforcement efforts). Specifically with respect to design, the procedural directive states the following:

> **DESIGN**
>
> *A wide range of options can serve to enhance bicycle and pedestrian mobility. Bicycle and pedestrian accommodation comes in many sizes and styles from signage and striping to sidewalks and shoulders. Context sensitive solution practices are encouraged to determine the appropriate solution for accommodating bicyclists and pedestrians within the project area so that they are consistent with local and regional transportation plans. Bicycle and pedestrian accommodations shall be integrated into the overall design process for state highway projects that begin the scoping process after the approval date of this procedural directive. Consideration of bicycle and pedestrian accommodations in on-going projects will be incorporated as reasonable and feasible given budget and schedule constraints.*
>
> *Current AASHTO and MUTCD standards for bicycle and pedestrian facilities shall be used in developing potential facility improvements. To provide consistent information on accommodating bicyclists and pedestrians on the state highway system, staff shall develop a chapter on bicycle and pedestrian design guidelines as part of the existing CDOT Design Manual.*

It is recognized that in some limited cases bicycle or pedestrian facilities may be impractical. Consequently the procedural directive provides the following:

> **EXEMPTION**
>
> *CDOT will utilize FHWA exemption guidance in situations where one or more of the following occur:*
>
> - *Bicyclists and pedestrians are prohibited by law from using the roadway*

- *The cost of establishing bikeways or walkways would be excessively disproportionate to the need or probable use. (Excessively disproportionate is defined as exceeding twenty percent of the cost of the larger transportation project.)*
- *Where scarcity of population or other factors indicate an absence of need.*

Requests for an exemption from the inclusion of bikeways and walkways shall be documented with supporting data that indicates the basis for the decision. Exemption requests shall be submitted to the Region Transportation Director and the headquarters Bicycle Pedestrian Coordinator. Review and response will be done within 30 days following submittal.

14.0.4 Design Exceptions

It is not the intent of this chapter to create a new process for documenting design variances and exceptions. A design letter will be used to document when any of the design criteria of this chapter cannot be met on a project. In addition to the Regional Transportation Director approval, when the exception is for a bicycle or pedestrian criteria, the headquarters Bicycle Pedestrian Coordinator must also acknowledge being provided an opportunity to comment on the request for an exception.

14.0.5 Federal Guidance Concerning Bicycle and Pedestrian Facilities

14.0.5.1 US Department of Transportation (DOT) Policy Statement

In a policy statement dated March 11, 2010, the US Secretary of Transportation stated the following:

> *The DOT policy is to incorporate safe and convenient walking and bicycling facilities into transportation projects. Every transportation agency, including DOT, has the responsibility to improve conditions and opportunities for walking and bicycling and to integrate walking and bicycling into their transportation systems. Because of the numerous individual and community benefits that walking and bicycling provide — including health, safety, environmental, transportation, and quality of life — transportation agencies are encouraged to go beyond minimum standards to provide safe and convenient facilities for these modes.*

And from Title 23 U.S.C. 217 the following is stated

> *Bicycle transportation facilities and pedestrian walkways shall be considered, where appropriate, in conjunction with all new construction and reconstruction of transportation facilities, except where bicycle and pedestrian use are not permitted.*

14.0.5.2 Restrictions on Severing Bicycle and Pedestrian Facilities

In addition to encouraging the provision of bicycle facilities, FHWA is prohibited from funding projects that would sever or have a significant adverse impact on the safety of non-motorized transportation. Title 23 of the United States Code includes the following (§109(m)):

> *Protection of Non-Motorized Transportation Traffic. --The Secretary shall not approve any project or take any regulatory action under this title that will result in the severance of an existing major route or have significant adverse impact on the safety for non-motorized transportation traffic and light motorcycles, unless such project or regulatory action provides for a reasonable alternate route or such a route exists.*

14.0.6 Context Sensitive Design

Context Sensitive Design applies to a transportation project's engineering design features, and may requires consideration of design features that help the project fit harmoniously into the surrounding. Context Sensitive Design is particularly relevant for pedestrian and bicycle related facilities because it balances the need to move cars with the priorities of the surrounding community.

14.0.7 User Counts

CDOT has a non-motorized traffic monitoring program to collect bicycle and pedestrian user counts. New or reconstruction projects, as well as facilities requiring non-motorized evaluation usage, should consider the installation of non-motorized continuous counting stations or conducting short duration counts.

By counting bicyclists and pedestrians, CDOT can obtain benchmark information on how many bicyclists and pedestrians there are on Colorado facilities. This information can be used in setting priorities for new facilities, making engineering decisions, and identifying potential routes. It can also measure increases in bicycling and walking as the Colorado network is improved. Additionally, counts provide a denominator for crash rates.

Coordination and support for selecting a site, purchasing counting equipment, and providing data are provided by CDOT's Traffic Analysis Unit (TAU) or Bicycle and Pedestrian Section within the Division of Transportation Development (DTD). When counting equipment is installed, the installation should be coordinated with DTD.

General specifications and guidance in for purchasing bicycle and pedestrian counting equipment can be obtained from DTD.

14.1 BICYCLE FACILITIES

Bicyclists should be expected on all of Colorado's state roadways except those where their use is prohibited. All design on CDOT facilities, except those roadways where cyclists are prohibited, shall include accommodations for bicyclists.

A map showing those roadways where bicyclists are prohibited is available on the internet at http://dtdapps.coloradodot.info/bike.

14.1.1 Accommodating Bicycles

Bicycle accommodations can take any number of forms. These most often include in-street facilities such as shared lanes, wide curb lanes, paved shoulders, bike lanes, or separated bike lanes. Separated shared use paths are another class of facility which may be provided for bicyclists.

When a corridor is being improved to accommodate bicyclists, the accommodation provided should be consistent to the maximum degree possible. Alternating facilities, such as from bike lanes to sidepaths back to bike lanes, can cause confusion for both bicyclists and motorists.

Roadway improvements for bicycles should be continued to logical termini. Where the improvement is a bike lane, bike route, or shared use path, advanced signage should be provided to inform bicyclists that the improvement is coming to an end.

14.1.1.1 Sharing Roadway Space

Bicycles operating on Colorado roadways are considered vehicles **(5)**. Consequently, bicyclists are subject to the same rules of the road as operators of other vehicles. The design criteria and treatment guidance provided in this chapter are intended to support the operation of bicycles as vehicles.

In-street facilities will be the most common facilities provided on CDOT roadway projects. In most cases the accommodation will be a bike lane or paved shoulder (See Section 14.1.3.5 below). If, however, this design chapter is applied on facilities that are not CDOT roadways, or if a project is constrained, other facilities may be appropriate. If a community or agency has adopted a minimum level of accommodation (level of service), bike lanes or shoulders that are wider than the minimums may be required to meet that level of accommodation. Where practical, the bicycle facility provided on CDOT roadway projects should comply with adopted bicycle plans.

14.1.1.2 Role of Design Factors

The level of accommodation for bicyclists can be measured by a number of methods ranging from subjective to objective. The 2010 *Highway Capacity Manual (HCM)* **(6)** now establishes an objective method for determining the level of bicycle accommodation (level of service) based upon the geometric and operational characteristics of the roadway being analyzed. This method is based upon numerous research projects which quantified what factors influence how bicyclists perceive a roadway's safety and comfort. The model for links (roadway segments between intersections) includes the following factors:

- Width of the outside through lane
- Presence and width of a paved shoulder or bike lane
- Encroachments into the bike lane
- Presence and width of a parking lane
- Percent of parking occupied by parked cars
- Pavement condition
- Operating speeds on the roadway
- Traffic volume on the roadway
- Percent heavy vehicles on the roadway

The primary geometric conditions that are influenced by design are the width of the outside lane, the presence of a paved shoulder or bike lane, the width of the paved shoulder or bike lane, and encroachments into the bike lane or shoulder. As stated above in Section 14.1.1.1, on new CDOT construction projects, it is likely that shoulders and bike lanes will be the facility of choice for accommodating bicycles. However, in some cases a shared lane, or wide outside through lane, may be adequate to accommodate bicyclists. On some projects pavement cannot be widened or restriped to provide shoulder or bike lane width. On these roads, the available roadway space and traffic conditions should be analyzed to determine if the minimum adopted level of service for bicycles can be achieved by adjusting lane widths to provide wide curb lanes.

14.1.1.3 The Bicycle as a Design Vehicle

As with the design of roadways, the design vehicle is an important consideration for bicycle facilities. Most design criteria for roadways, beyond the addition of extra space for the bike lane or paved shoulder, will not be impacted by the bicycle as a design vehicle. On a shared use path, the bicycle and other non-motorized users are used as design vehicles. Their characteristics dictate numerous design values and criteria such as design speeds, stopping sight distances, maximum degree of horizontal curvature, minimum vertical curve lengths, etc. The design values used in this chapter are based upon those in the AASHTO *Guide for the Development of Bicycle Facilities* **(2)**, with supplemental information provided from the FHWA *Characteristics of Emerging Road and Trail Users and Their Safety* **(7)**.

Design vehicle considerations can be grouped as key dimensions, operating space, and key performance criteria. These are briefly summarized in the following paragraphs.

The key dimensions that are associated with the various types of bicycles are listed in Table 14-1. These are not exact and represent the 85th percentile (unless otherwise noted) of distribution that encompasses most bicyclists.

Recommended widths of bicycle facilities can be determined from the bicyclist operating space, as shown in Figure 14-1. Additional operating width may be required in unique circumstances including but not limited to steeper grades, mixed traffic (parked cars), and poorly lit areas.

Key performance criteria that are associated with the various types of bicycles are listed in Table 14-2. These performance criteria vary greatly based on a number of factors including age, health, physical and cognitive abilities, bicycle design, traffic, environmental conditions, and terrain.

User Type	Feature	Dimension
Typical upright adult bicyclist	Physical width (95^{th} Percentile)	30 in.
	Physical length	70 in.
	Physical height of handlebars (typical dimension)	44 in.
	Eye height	60 in.
	Center of gravity (approximate)	33-44 in.
	Operating width (minimum)	48 in.
	Operating width (preferred)	60 in.
	Operating height (minimum)	100 in.
	Operating height (preferred)	120 in.
Recumbent bicyclist	Physical Length	82 in.
	Eye height	46 in.
Tandem bicyclist	Physical length (typical dimension)	96 in.
Bicyclist with child trailer	Physical width	30 in.
	Physical length	117 in.
Hand bicyclist	Eye height	34 in.
Inline skater	Sweep width	60 in.

Table 14-1 Key Dimensions of Bicycles

Bicyclist Type	Feature	Value
Typical upright adult bicyclist	Speed, paved level terrain	8 - 15 mph
	Speed, downhill	20 - 30 plus mph
	Speed, uphill	5 - 12 mph
	Perception reaction time	1 - 2.5 seconds
	Acceleration rate	1.5 - 5 ft/s^2
	Coefficient of friction for braking, dry level pavement	0.32
	Deceleration rate (dry level pavement)	15 ft/s^2
	Deceleration rate for wet conditions (50-80% reduction in efficiency)	8 - 10 ft/s^2
Recumbent bicyclist	Speed, level terrain	11 - 18 mph
	Acceleration rate	3 - 6 ft/s^2
	Deceleration rate	10 - 13 ft/s^2

Table 14-2 Key Performance Criteria

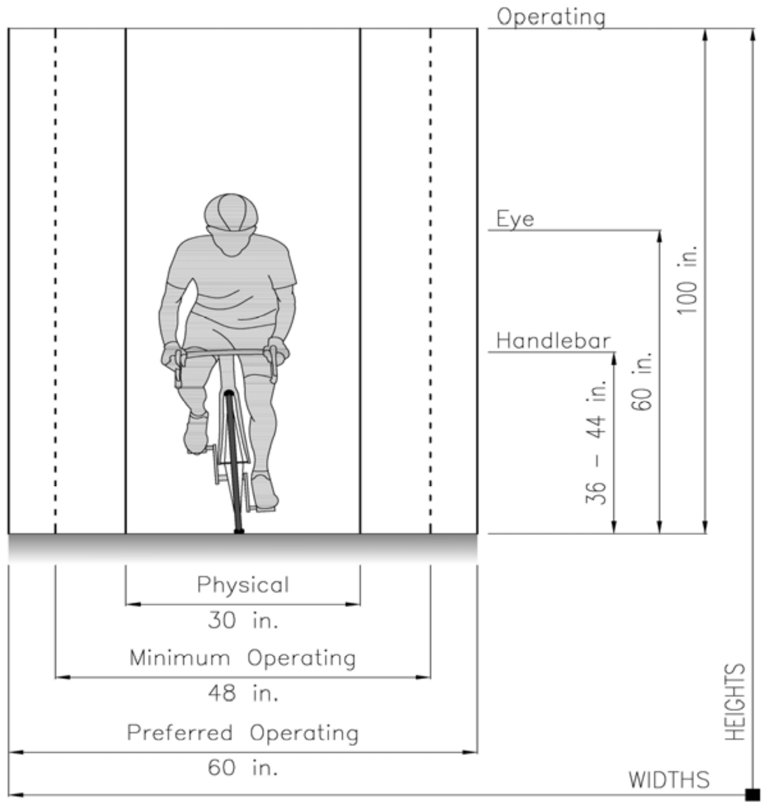


Figure 14-1 Bicycle Operating Space Requirements

With regard to calculated design values such as stopping sight distance or the minimum length of vertical curves, the equations used to calculate the design values are the same for non-motorized operators as they are for motorized vehicles. Appropriate assumptions and input values will be provided in the chapter section related to specific design values (Section 14.2.3.3).

14.1.2 Bike Routes

Bike routes are not an actual facility type. A bike route is a designation of a facility, or collection of facilities, that links origins and destinations that have been improved for, or are considered preferable for, bicycle travel. Bike routes include a system of wayfinding and route signs that provide at least the following basic information:

- Destination of the route
- Distance to the route's destination
- Direction of the route

Bike routes can be designated in two ways: General Routes and Number Routes. General Routes are links with a single origin and a single destination. Number Routes form a network of bike routes that connect several origins to several destinations.

14.1.2.1 General Bike Routes

General Routes connect users to destinations within a community. Typical destinations include the following:

- Attraction Areas (i.e. stadiums, parks, etc.)
- Neighborhood Areas (i.e. downtown, historic neighborhoods, etc.)
- Trail Networks or trailheads (i.e. Glenwood Canyon Trail)

BICYCLE GUIDE signs may be provided along designated bicycle routes to inform bicyclists of bicycle route direction changes and to confirm route direction, distance, and destination. Typical signs that convey the basic wayfinding information for general routes are shown below in Figure 14-2. The *MUTCD* provides a number of different types of signs that can be used to provide guidance along bike routes. Some of these are shown below.

Figure 14-2 Examples of BICYCLE GUIDE Signs

14.1.2.2 Numerically Labeled Bike Routes

Some communities may implement a numerically labeled system of bike routes. These routes should be designated using BIKE ROUTE signs (Figure 14-3). BICYCLE ROUTE signs can be customized by adding a specific community logo in the upper portion of the ellipse.

Figure 14-3 Examples of BIKE ROUTE Signs

A subset of numerically labeled bike routes is the U.S. Bicycle Route system. Where a designated bicycle route extends through two or more states, a coordinated submittal by the affected states for an assignment of a U.S. Bicycle Route number designation is sent to the American Association of State Highway and Transportation Officials (AASHTO) **(8)**. A system of proposed U.S. Bicycle Routes is being developed. Colorado has not yet defined its U.S. Bicycle Routes; however, the AASHTO task force leading this effort has proposed several corridors through Colorado. For these routes the U.S. BIKE ROUTE (Figure 14-4) sign should be used to designate the routes.

M1-8a

Figure 14-4 U.S. BIKE ROUTE Sign

14.1.3 Shared lanes

A *shared lane* is a lane of a traveled way that is open to bicycle travel and vehicular use. In this *Roadway Design Guide* it refers to a lane of less than 14 feet in width. Lanes 14 feet wide or wider are considered *wide curb lanes*.

The *Highway Capacity Manual* method can be used to determine what accommodations are necessary to meet a minimum level of accommodation for bikes along a bike route. On local roadways with low volumes and speeds, a shared lane may be all that is needed to comfortably accommodate bicyclists. On other roadways, a higher level of accommodation might be desirable; however, it may be infeasible to provide bike lanes or paved shoulders, or to adjust lane widths to provide a wide curb lane. In these latter cases the following potential traffic control devices could be considered, particularly if the roadways are identified as priority routes in an adopted bicycle plan:

14.1.3.1 Bicycle May Use Full Lane Sign (R4-11)

The BICYCLE MAY USE FULL LANE sign (R4-11) may be used on roadways where the lanes are too narrow for bicyclists and motorists to operate side by side within a single lane **(9)**. On roadways with significant volumes, following motorists would likely be delayed while waiting for a gap to pass the bicyclist. On such roadways, the BICYCLE MAY USE FULL LANE sign should be considered to inform users that bicyclists have the legal right to claim the lane if the right-hand lane available for traffic is not wide enough to be safely shared with motor vehicles **(10)**. Guidance on the BICYCLE MAY USE FULL LANE sign is provided in the *MUTCD*.

Figure 14-5 Bicycles May Use Full Lane Sign

A SHARED LANE MARKING (see Section 14.1.2.2.1) may be used in conjunction with the BICYCLES MAY USE FULL LANE sign.

14.1.3.2 SHARE THE ROAD Sign Assembly (W11-1 + W16-1P)

In situations where there is a need to warn drivers to watch for bicycles traveling along the highway, the SHARE THE ROAD sign assembly may be considered (see Figure 14-6).

The SHARE THE ROAD sign assembly may be installed on State-maintained roadways at the discretion of each region's Traffic Engineer. To have maximum effect, these signs should be used with discretion. Consideration for placement should be given where:

- A relatively high number of cyclists can be expected on the roadway
- The roadway cannot be improved for cyclists
- The road narrows for a short distance and a motorist and bicyclist may unexpectedly find themselves using the same roadway such as at the end of a bike lane or bridge approach
- There has been a significant history of bicycle crashes.

In addition to these reasons, the Share the Road sign assembly may be appropriate where ***(11):***

- Designated bicycle trails that are placed on short stretches of a major roadway that has not been improved for bicycling
- Roadway where a known conflict problem exists
- Roadway sections adjacent to shared use paths where some bicyclists choose to ride on the roadway

Figure 14-6 SHARE THE ROAD Sign Assembly

On approaches to bridges, tunnels, or any other section where motorists and bicyclists have reduced sight distance or where operating widths must be less than desirable due to right-of-way or actual roadway geometry restrictions, a SHARE THE ROAD assembly may be appropriate. In these cases consider adding flashing beacons to the assembly that can be either actively or passively triggered by bicyclists. The duration of the flashing beacon's activation should be such that a motorist passing the active flashing beacon will be likely to pass bicyclists who activated the treatment within the area of limited sight distance. This duration can be calculated using the following equation:

$$t_f = 1.47\left(\frac{l_c}{S_b} - \frac{l_c}{S_m}\right)$$

Where

t_f = duration of flashing (sec)

l_c = length of constrained area (ft)

S_b = speed of bicyclist (mph)

S_m = speed of motorists (mph)

The recommended assumed speed of the bicyclist on flat terrain for this application is 10 mph. This is the observed average speed of bicyclists **(7)**. Adjustments for grade should be made, particularly on uphill sections, where bicyclists will be traveling slower than average speeds.

A SHARED LANE MARKING (see Section 14.1.2.2.3) may be used in conjunction with the SHARE THE ROAD sign assembly.

14.1.3.3 Shared Lane Markings

SHARED LANE MARKINGS (Figure 14-7) are intended to perform any of several functions **(12)**:

- Assist bicyclists with lateral positioning in a shared lane with on-street parallel parking in order to reduce the chance of a bicyclist impacting the open door of a parked vehicle
- Assist bicyclists with lateral positioning in lanes that are too narrow for a motor vehicle and a bicycle to travel side by side within the same traffic lane
- Alert road users of the lateral location bicyclists are likely to occupy within the traveled way
- Encourage safe passing of bicyclists by motorists
- Reduce the incidence of wrong-way bicycling

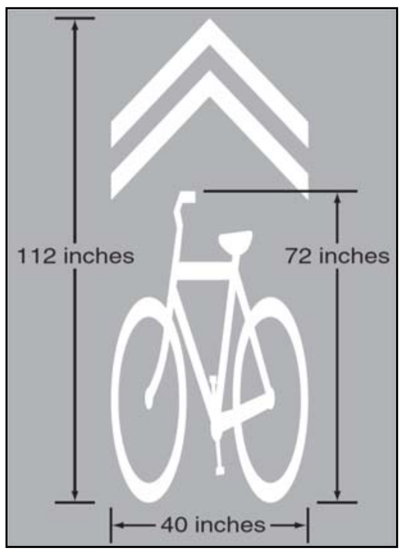


Figure 14-7 SHARED LANE MARKING

Refer to the *MUTCD* for proper placement of SHARED LANE MARKINGS.

SHARED LANE MARKINGS are not intended as a replacement for bike lanes. They should not be considered such even on constrained facilities. On higher speed roadways (> 35 mph) they may not be as effective as on lower speed roadways, bike lanes should be provided instead. If used on a bike route, additional improvements such as traffic calming or signal improvements should be considered for implementation in conjunction with SHARED LANE MARKINGS.

14.1.4 Wide Curb Lanes

In restricted urban conditions, where it is not possible to include bike lanes or paved shoulders or on lower volume, lower speed collector streets, a wide curb lane can help accommodate both bicycles and motor vehicles in the same lane. The *Highway Capacity Manual* (HCM) established methods can be used to identify the minimum wide curb lane width that will meet a target level of accommodation. Fourteen feet is the recommended minimum lane width for a wide curb lane, and within which a motorist may safely pass a bicyclist without encroaching into an adjacent lane.

The SHARED LANE MARKING and/or SHARE THE ROAD assembly may be used in wide curb lanes.

14.1.5 Paved Shoulders

Including paved shoulders during roadway construction, adding paved shoulders to an existing roadway without curb and gutter, or restriping a roadway to obtain a paved shoulder outside the travel lane can be an effective and relatively inexpensive way to improve a roadway for bicyclists. Gravel shoulders are not acceptable as bicycle facilities. Adding or widening of paved shoulders may be subject to Municipal Separate Storm Sewer System (MS4) permitting requirements which could substantially increase retrofit costs.

To accommodate bicyclists, paved shoulders at least 4 feet wide should be provided. Table 4-1 Geometric Design Standards (in Chapter 4) provides CDOT's minimum standard shoulder widths.

14.1.5.1 Additional Width

Some jurisdictions may have adopted a minimum paved shoulder width above those required for Type C or D roadways (as shown in Figures 4-1 through 4-4, in Chapter 4) within their bicycle master plans. When these local shoulder widths exceed the planned or typical CDOT shoulder for this type of location, the project manager should consider accommodating local requirements when additional funding is provided by the local community to supplement the available budget.

Other communities or agencies may have adopted a minimum bicycle Level of Service that is to be met on their roadways. CDOT projects within these jurisdictions should be designed to meet the adopted minimum bicycle Level of Service unless the available budget prohibits this action. Table 14-3 uses the aforementioned *HCM* method to provide the maximum design daily traffic for which a given shoulder width can provide a given bicycle Level of Service. For a given speed limit, percent heavy vehicles, and shoulder width, Table 14-3 provides the maximum number roadway AADT that will provide a selected bicycle Level of Service.

Scenic Byways plans for roadways may also specify wider shoulders. These plans should be accommodated during design.

Adopted Bicycle Level of Service = B

	Speed Limit (or Design Speed) 35						Speed Limit (or Design Speed) 45					
	Percent Heavy Vehicles						Percent Heavy Vehicles					
Shoulder width, ft	2	4	6	8	10	12	2	4	6	8	10	12
4	13300	7500	4500	3600	3100	2700	11200	6200	3900	3400	3000	2500
6		26400	10100	4800	3700	3200		16400	6600	4200	3500	3000
8				27000	8100	3700				12200	3900	3400

	Speed Limit (or Design Speed) 55						Speed Limit (or Design Speed) 65					
	Percent Heavy Vehicles						Percent Heavy Vehicles					
Shoulder width, ft	2	4	6	8	10	12	2	4	6	8	10	12
4	9900	5600	3800	3300	2800	2400	8900	5200	3700	3200	2700	2300
6		12200	6100	3900	3400	2800	29900	10300	5600	3800	3300	2800
8			29900	7600	3800	3300			22400	5200	3800	3200

Adopted Bicycle Level of Service = C

	Speed Limit (or Design Speed) 35						Speed Limit (or Design Speed) 45					
	Percent Heavy Vehicles						Percent Heavy Vehicles					
Shoulder width, ft	2	4	6	8	10	12	2	4	6	8	10	12
4			12700	5100	3700	3100		21200	7100	4400	3500	2900
6				24900	7300	3700				11600	3900	3400
8											22400	4700

	Speed Limit (or Design Speed) 55						Speed Limit (or Design Speed) 65					
	Percent Heavy Vehicles						Percent Heavy Vehicles					
Shoulder width, ft	2	4	6	8	10	12	2	4	6	8	10	12
4		15800	6500	4100	3400	2800		12700	6100	3900	3200	2700
6			27600	7100	3800	3200				5200	3700	3100
8					12000	3800					7600	3600

Adopted Bicycle Level of Service = D

	Speed Limit (or Design Speed) 35						Speed Limit (or Design Speed) 45					
	Percent Heavy Vehicles						Percent Heavy Vehicles					
Shoulder width, ft	2	4	6	8	10	12	2	4	6	8	10	12
4					9300	3700				14700	4100	2900
6						13900					20700	4400
8						15100						

	Speed Limit (or Design Speed) 55						Speed Limit (or Design Speed) 65					
	Percent Heavy Vehicles						Percent Heavy Vehicles					
Shoulder width, ft	2	4	6	8	10	12	2	4	6	8	10	12
4				9000	3900	3200			26200	6200	3800	3100
6					11300	3700					7100	3500
8						16600						9500

Table 14-3 Maximum motor vehicle service volumes for given Bicycle LOS grades

Notes:

Volumes are based upon a two-lane roadway. For maximum service volumes on a four-lane or six-lane roadway double or triple the values accordingly.

Values are established using the *HCM* methodology for roadway links.

Table assumes the following:

K = 0.10 D= 0.53 PHF = 1 PavCon = 4 outside lane width = 12 feet

14.1.5.2 Shoulders on Steep Grades

The additional effort required of bicyclists riding uphill frequently results in their having a greater side-to-side sweep width than those riding on a flat roadway. A bicyclist riding downhill may also need additional space to maintain a comfortable distance from the edge of the pavement and potential adjacent motorists. Consequently, on roadways with significant grades, or long grades, shoulders of 6 feet or greater width should be provided.

14.1.5.3 Rumble Strips

Where appropriate, rumble strips should be installed per CDOT Standard Plan No. M-614-1. On roadways identified as bicycle routes continuous rumble strips shall not be used. Rumble strips shall not be installed on shoulders less than 6 feet wide when guardrail is placed at the edge of the shoulder.

Rumble strips should be placed as closely as possible to the right edge of the roadway edge line. A minimum of 4 feet clear shoulder should be provided to the right of the rumble strips. A warning marking as shown in Figure 14-8 should be placed in advance of each rumble strip installation.

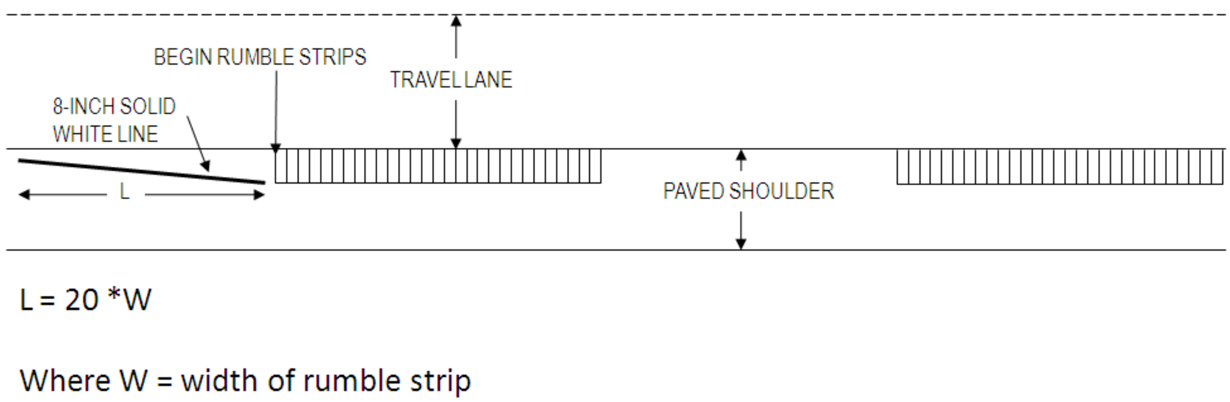


Figure 14-8 Advance Warning Stripe for Rumble Strips

14.1.5.4 Shoulders at Intersections

At intersections with right-turn lanes, a paved shoulder is typically continued along the outside of the right turn lane. Some through bicyclists may continue to ride along the shoulder even though it compromises their safety at the intersection. Consequently, a 4-foot minimum space (bike slot) should be striped between the right-turn lane and the through lanes. This is illustrated in Figure 14-9.

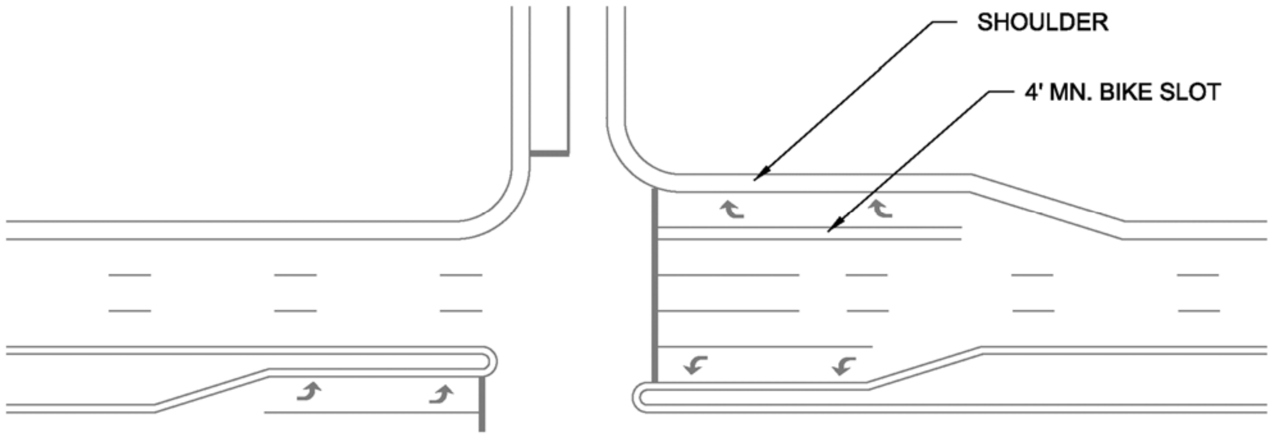


Figure 14-9 Bike slot at intersection.

14.1.6 Bike Lanes

Bike lanes are lanes that have been designated with pavement markings for the preferential use of bicyclists. They are typically one-way facilities located to the right of the general travel lanes on both sides of two-way streets. They may be placed on the left side of one-way streets if predominant travel paths or conflict points suggest this is a desirable option.

14.1.6.1 Bike Lane Width

The minimum bike lane width on a roadway with no curb and gutter is 4 feet. On roadway with curb and gutter, the minimum width of a bike lane is five feet measured from the face of curb. If a 2-foot gutter is used a 6-foot bike lane measured to the face of curb is recommended. As with paved shoulders (Section 14.1.2.5), adopted bicycle plans and Scenic Byway plans should be consulted to determine if wider bike lanes are specified or if a wider bike lane is needed to meet an adopted Level of Service standard.

On roadways with narrow parking lanes, wider bike lanes (six or seven feet wide) should be considered. This allows more space for bicyclists to avoid potential opening car doors. On roadways with on-street parking where there is high parking turnover 13 feet minimum is recommended between the face of curb and the left side of the bike lane.

On roadways where significant volumes of bicyclists are expected, creating a potential need for passing maneuvers, six- or eight-foot bike lanes should be considered.

Wide shoulders or bike lanes may be interpreted by motorists as additional general purpose travel lanes or parking lanes. This can be discouraged through the use of designated or buffered bike lanes (Section 14.1.6.5).

As with paved shoulders, additional width should be considered on roadways with significant or long grades. Another option on significant grades is to remove the bike lane on the downhill side of the road, reducing but not eliminating the shoulder, and to install BICYCLE MAY USE FULL LANE signs (R4-11) and SHARED LANE MARKINGS. The additional space gained from removing the bike lane on the downhill side of the road should be used to increase the bike lane width on the uphill side of the road.

14.1.6.2 Designating Bike Lanes

Bike lanes shall be designated with the bicycle symbol with the directional arrow being optional (Figure 14-10). Although using the directional arrow is optional, it's strongly encouraged to better communicate the requirement for bicyclists to ride with traffic as the law requires.

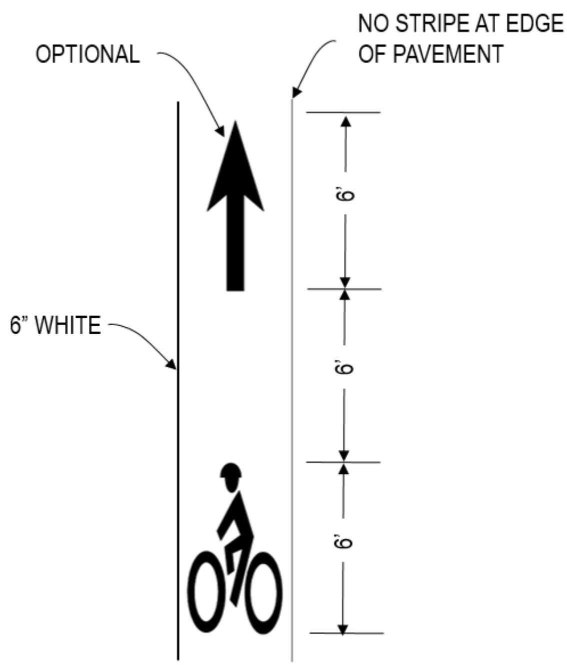


Figure 14-10 Detail of Bike Lane Designation

Bicycle lane markings should be placed after intersections and major driveways. In rural areas the maximum spacing of bike lane markings should not exceed 1320 feet. In urban areas the spacing should not exceed 600 feet.

The 6 inch white stripe on the left of the bike lane should become a dotted (2-foot line with a 4-foot gap) at improved bus stops with alighting pads to clarify that buses are to move right to allow transit riders to disembark off of the roadway.

14.1.6.3 Contraflow Bike Lanes

A contraflow bicycle lane is an area of the roadway designated to allow bicyclists to travel in the opposite direction of traffic on a roadway that restricts motor vehicle travel to one direction.

These may be used to make convenient connections for bicyclists along otherwise one-way streets. If used, a contraflow bicycle lane should be marked so that bicyclists in the contraflow lane travel on their right-hand side of the road.

Where used, a contraflow bicycle lane shall be separated from opposite-direction travel by use of a solid double yellow center line marking, or a painted or raised median island (Figure 14-11).

The minimum contraflow bike lane width on a roadway with no curb and gutter is 4 feet. On roadway with curb and gutter, the minimum width of a contraflow bike lane is 5 feet measured from the face of curb. If a 2-foot gutter is used a 6 foot bike lane measured to the face of curb is recommended.

Where intersection traffic controls along the street exist (e.g., stop signs, flashing light signals or traffic signals) appropriate devices shall be oriented toward bicyclists in the contraflow lane. At speeds greater than 40 mph, a raised separator or painted buffer area should be used to separate the contraflow bicycle lane from the opposing travel lanes. At locations where a contraflow bicycle lane is provided across an intersection or a driveway entrance, pavement markings that inform intersection or driveway traffic of the presence of the bicycle facility and the direction of permitted bicycle traffic may be placed within the contraflow bicycle lane across the intersection or driveway opening.

ONE WAY (R6-1 or R6-2) signs should not be used where signs are provided to regulate turns from streets or driveways that intersect with a roadway that has a contraflow bicycle lane. TURN PROHIBITION signs (R3-1 or R3-2) with a supplemental message EXCEPT BICYCLES (or the word EXCEPT over the bicycle symbol) plaques should be used. If DO NOT ENTER signs (R5-1) are used, an EXCEPT BICYCLES plaque should be placed under the DO NOT ENTER sign.

A bicycle lane for travel in the same direction as the general purpose lanes may be relocated from the right side of the roadway to the left side of the general purpose travel lanes.

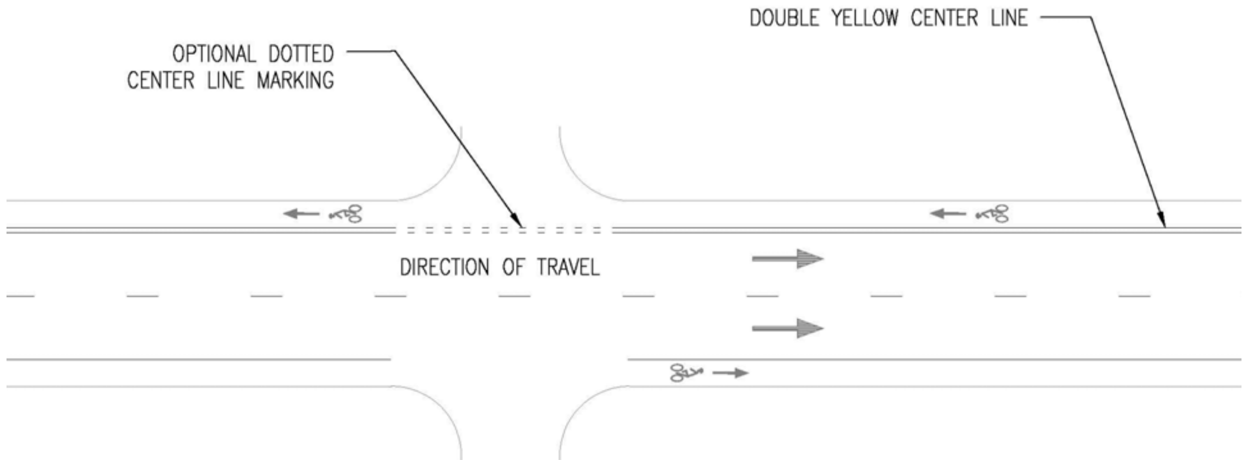


Figure 14-11 Example Contraflow Bicycle Lane Markings

14.1.6.4 Bike Lanes at Driveways and Intersections

In Colorado, bicycles are vehicles and are required to follow the rules of the roadway when riding on the street **(5)**. Consequently, the striping and marking of bike lanes at intersections should support the operations of bicycles as vehicles, and the safe mixing of bicyclists with motorists at conflict points such as driveways and intersections.

Bicyclists are required to ride on the right hand side of the rightmost lane that is intended for the direction they are traveling. Bicyclists may use left and right turn lanes when making the respective movements. Bicyclists are not required to ride at the right edge of the pavement; they may move left when passing slower vehicles, to make a left turn, or to avoid debris or obstacles on the pavement **(10)**.

For both motor vehicles and bicycles the approach to a right turn and a right turn shall be made from as close as practicable to the right-hand curb or edge of the roadway **(14).** Prior to moving into a bike lane to make a right turn, motorists must yield to bicyclists who. To support crossing a bike lane at a right turn the bike lane striping is either terminated or becomes dotted on the approach to the intersection. The purpose of a solid white line is to discourage motorists from crossing the line. Changing the line pattern to a dotted line makes the striping appropriate for the required behaviors **(15)**. It also informs the bicyclists that they are entering a potential conflict area. The length of the dotted line can be varied based upon the speed of the approaching roadway. A minimum 50-foot dotted line (or gap in the bike lane) should be provided; this is based upon a 1:12 taper rate, and a 4-foot bike lane. An 18:1 taper rate or 24:1 taper rate (75-ft and 100-ft) or longer dotted length of bike lane can be used on higher speed roadways.

When motorists cross a bike lane to move into a right turn lane, motorists are required to yield the right of way to bicyclists in the bike lane **(21)**. This means the use of the BEGIN RIGHT TURN LANE YIELD TO BIKES sign (R4-4) is appropriate when it’s added to a roadway where a turn lane is developed (Figure 14-13, Figure 14-17, Figure 14-18, and Figure 14-20). However, in the trap lane condition (Figure 14-15), the through bicyclists must cross the motorists' path to continue through the intersection. In this case the bicyclists must yield to the motorist before moving left; therefore the R4-4 is not appropriate in these conditions.

On retrofit projects, it may not be possible to include bike lanes through existing intersections with turn lanes. On such projects the bike lane should be terminated in advance of the intersection and SHARED LANE MARKINGS should be considered for the left side of the right turn lane. An example of this marking is shown in Figure 14-26 in the buffered bike lanes section.

In locations with significant numbers of right turning bicyclist, an additional bike lane for right turning bicyclist can be provided. The installation of right turn bike lanes may be considered at high volume high speed right turn lanes. These bike lanes should include right turn arrows and the text message ONLY.

By riding in the roadway in a predictable and consistent manner bicyclists are more visible to motorists. This increased visibility has been shown to reduce crashes when compared to riding on a sidewalk or pathway next to the roadway **(16, 17, 18, 19, 20)**.

14.1.6.4.1 Bike Lanes at Continuous Flow Intersections

At continuous flow intersections a bike lane is provided for through bicyclists. Two options are available for left turning bicyclists:

- Left turning bicyclists may ride through the intersection or in the left turn lanes. Additional bike lanes for left turning cyclists may be considered.
- Left turning bicyclists may make two consecutive through movements obeying all traffic control devices **(23).** A staging area for the bicyclists to wait between through movements should be provided for bicyclists making this maneuver.

Dedicated right turn lanes for bicyclists should be considered at continuous flow intersections.

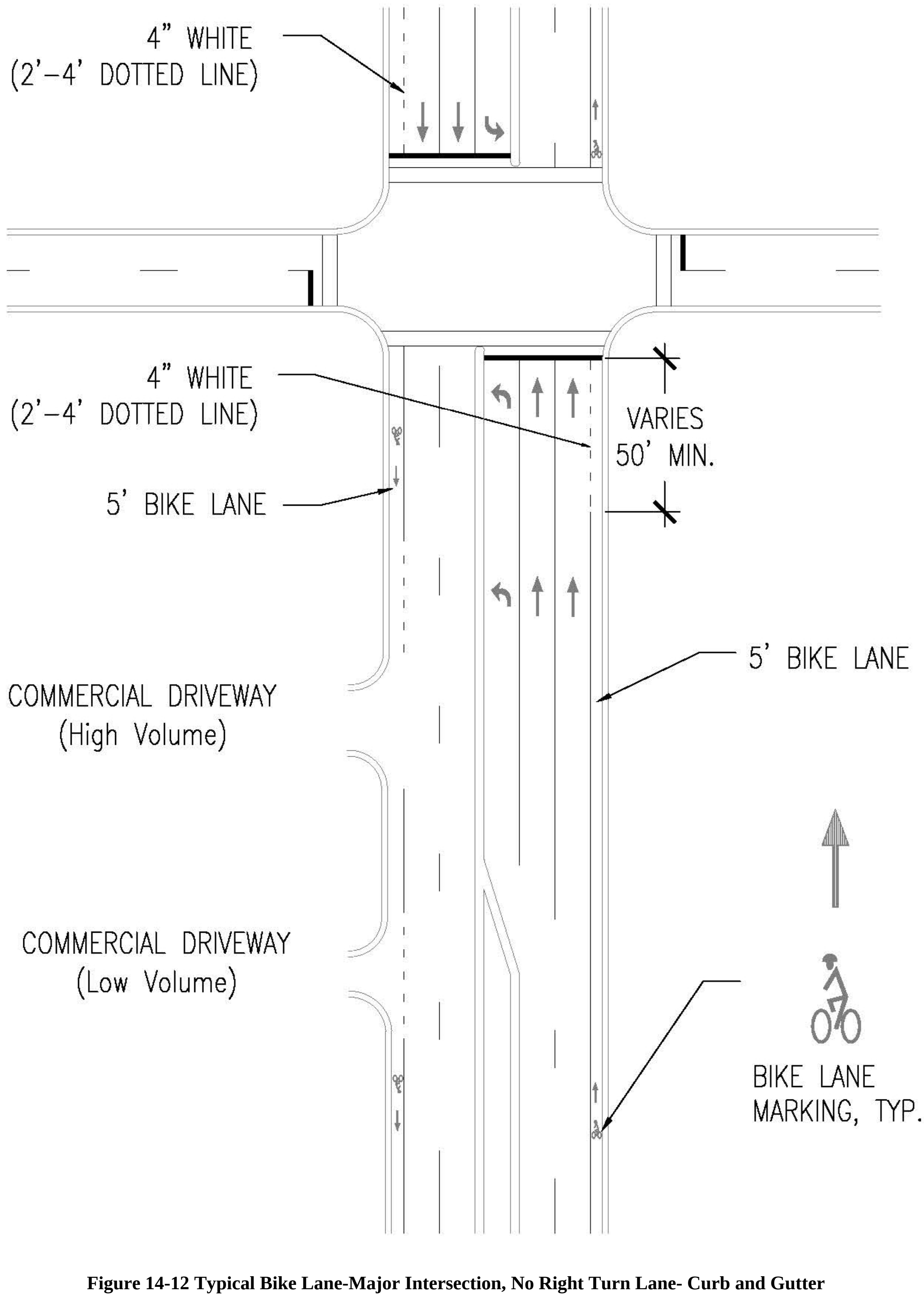


Figure 14-12 Typical Bike Lane-Major Intersection, No Right Turn Lane- Curb and Gutter

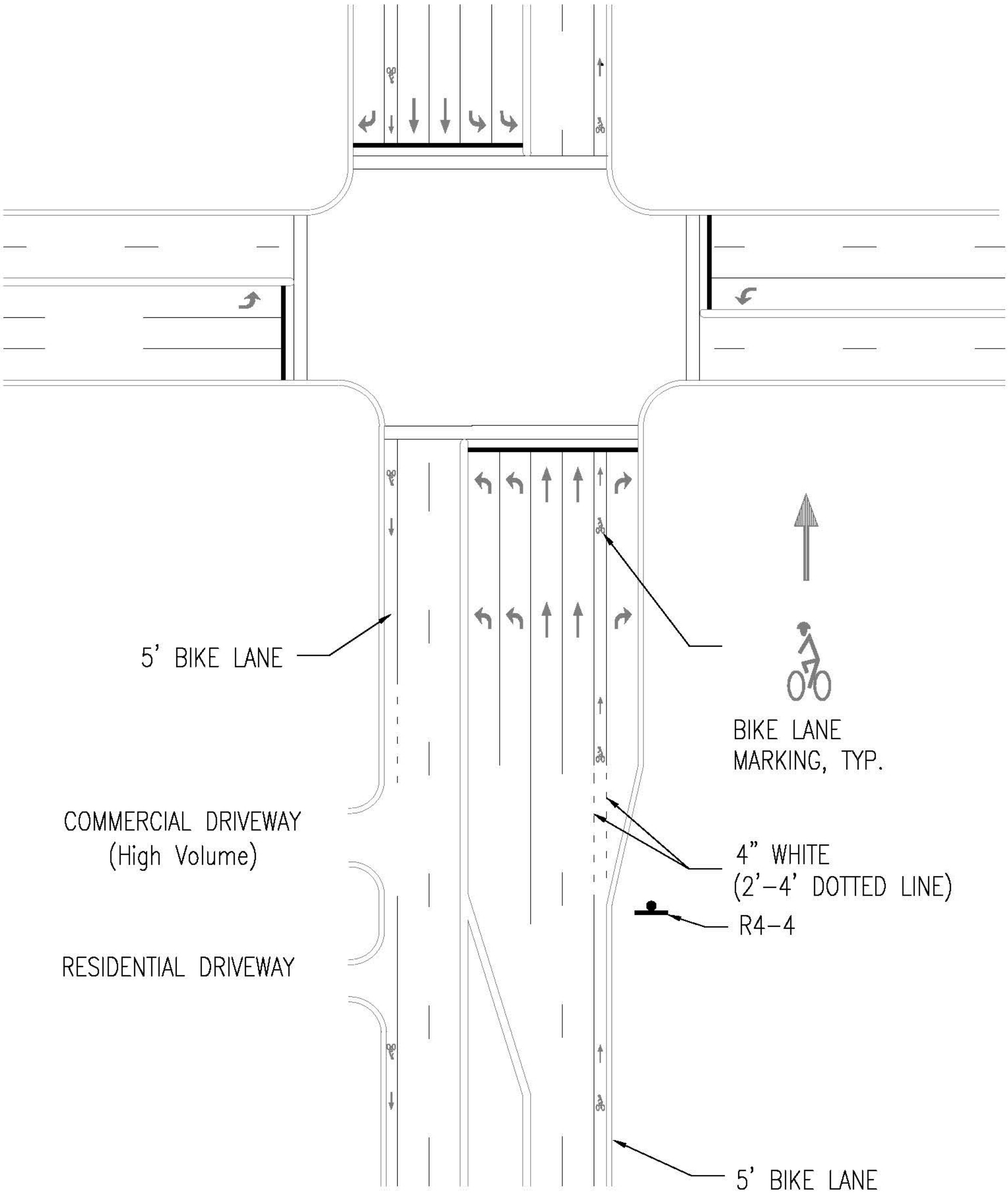


Figure 14-13 Typical Bike Lane-Major Intersection. Right Turn Lane

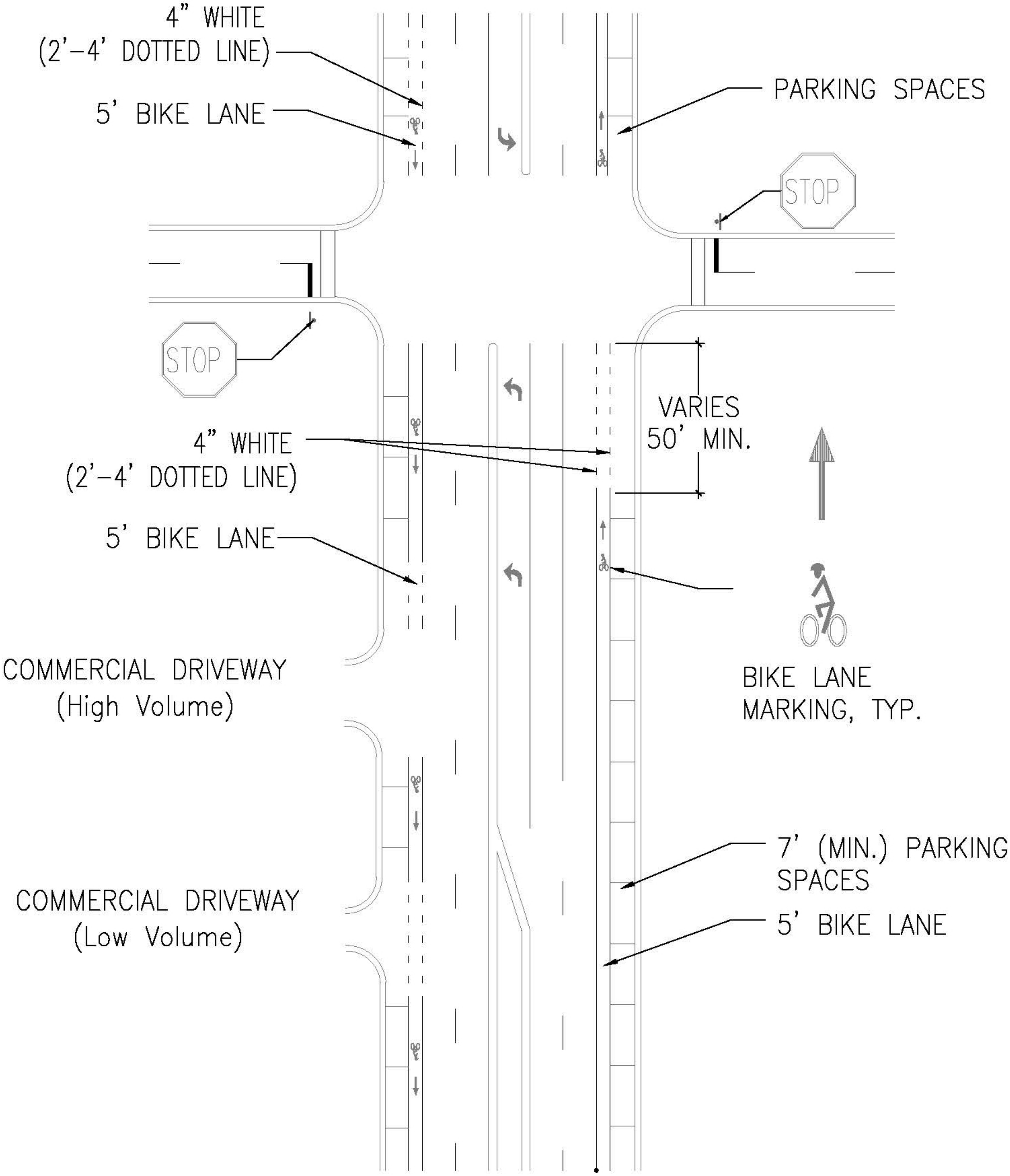


Figure 14-14 Typical Bike Lane - Major Intersection, No Right Turn Lane, On-Street Parking

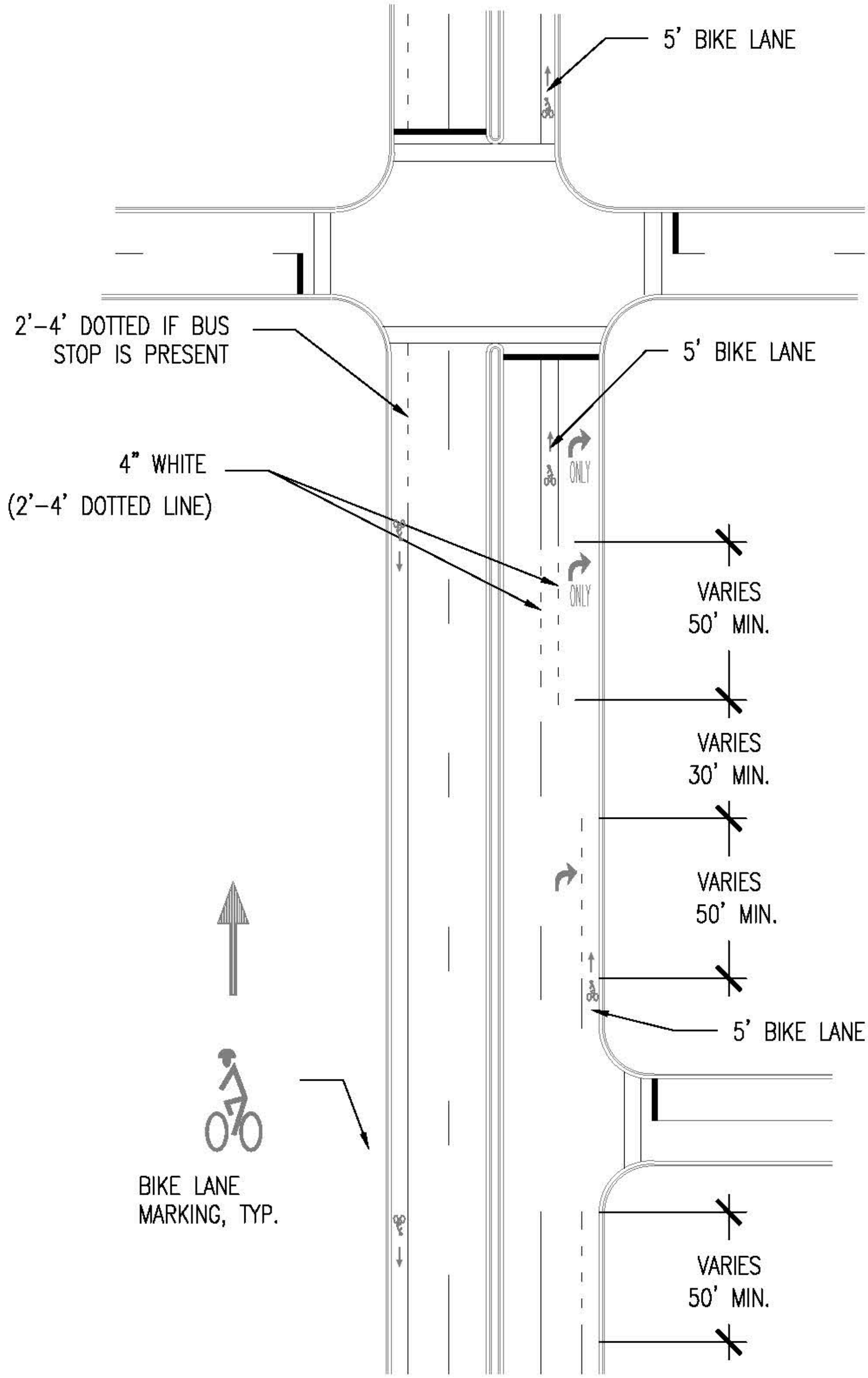


Figure 14-15 Typical Bike Lane-Major Intersection, Right Turn Trap Lane-Bus Stop

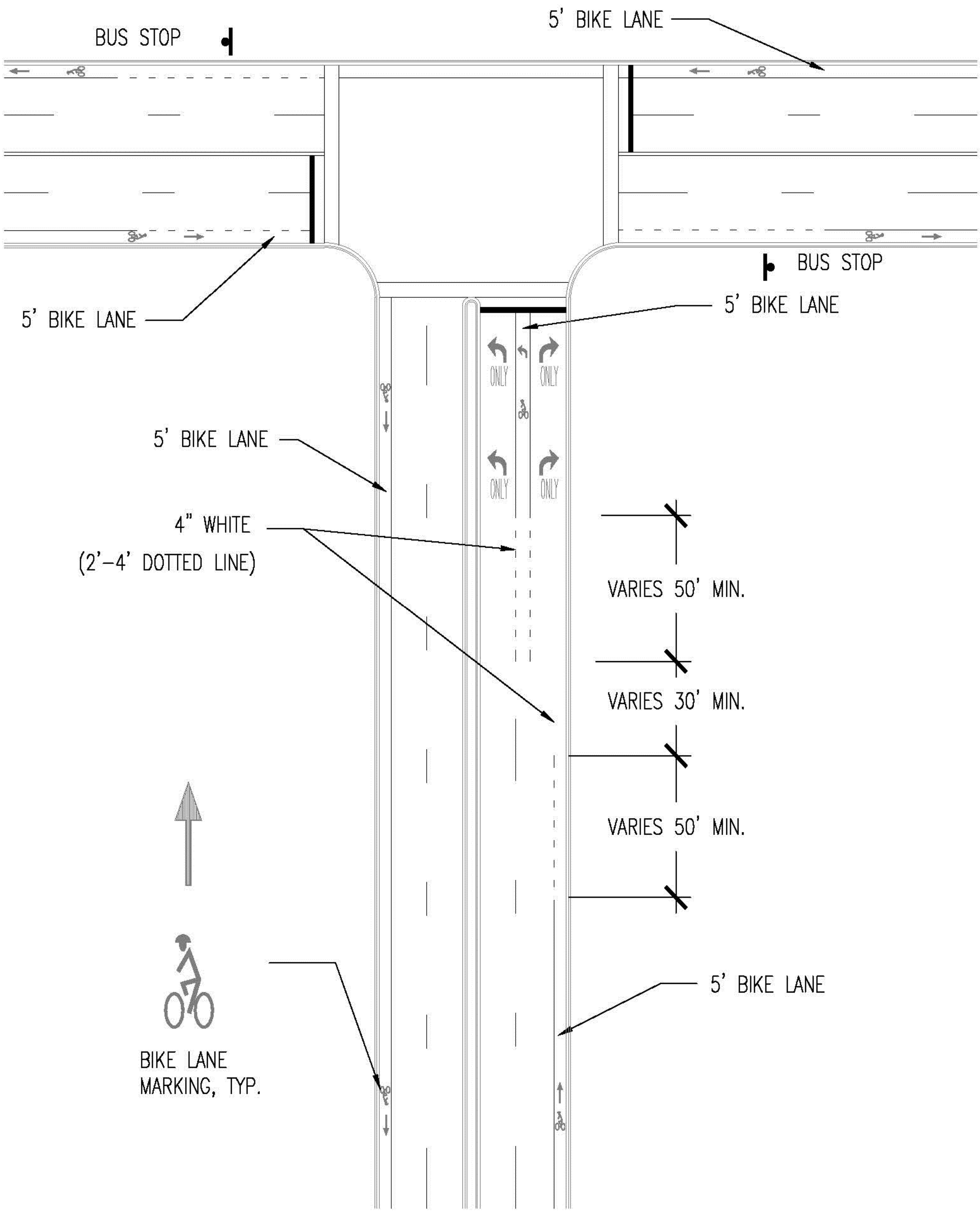


Figure 14-16 Typical Bike Lane-Tee Intersection. Right Turn Must Turn Right-Bus Stop

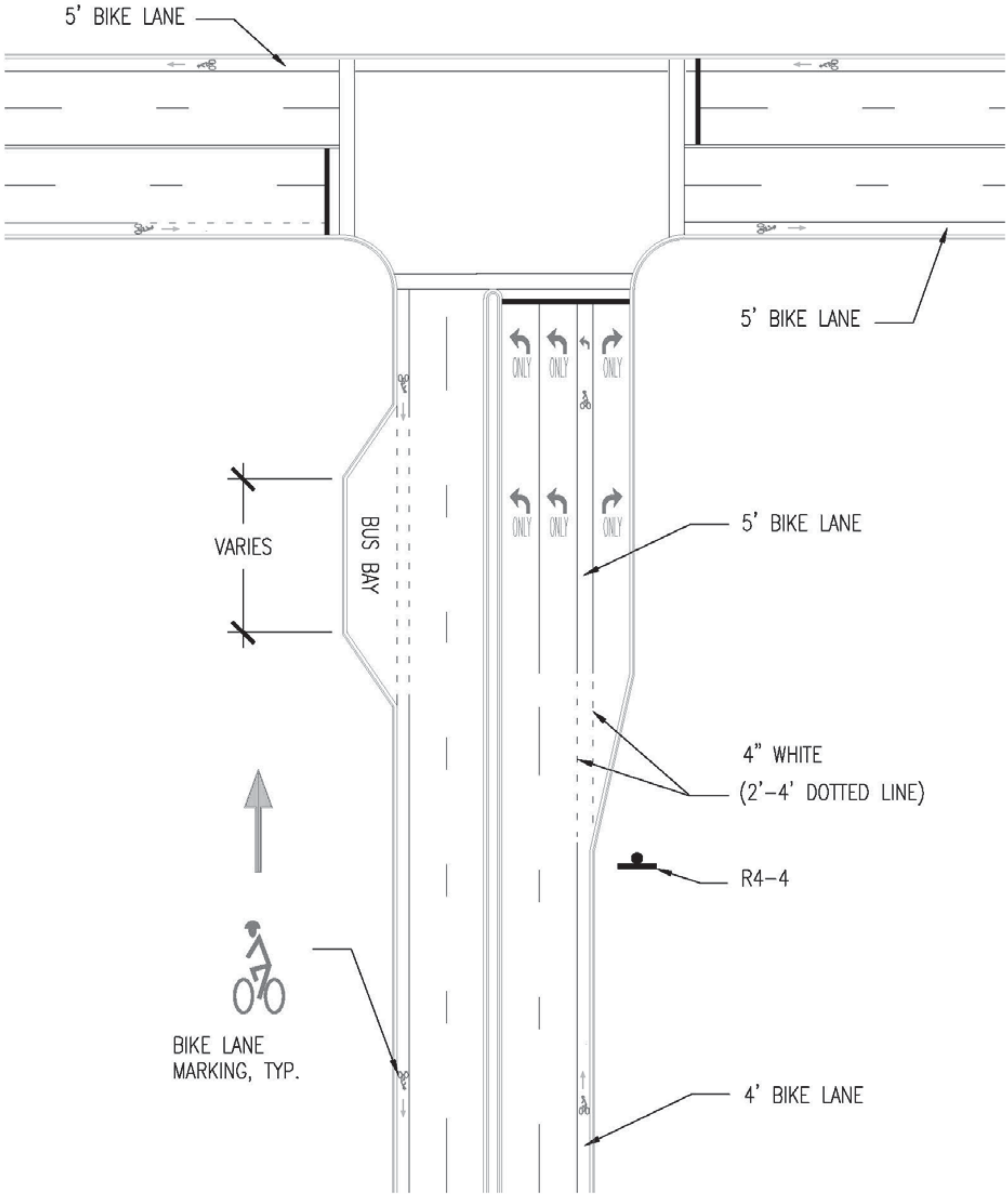


Figure 14-17 Typical Bike Lane-Tee Intersection. Right Turn Lane-Bus Bay

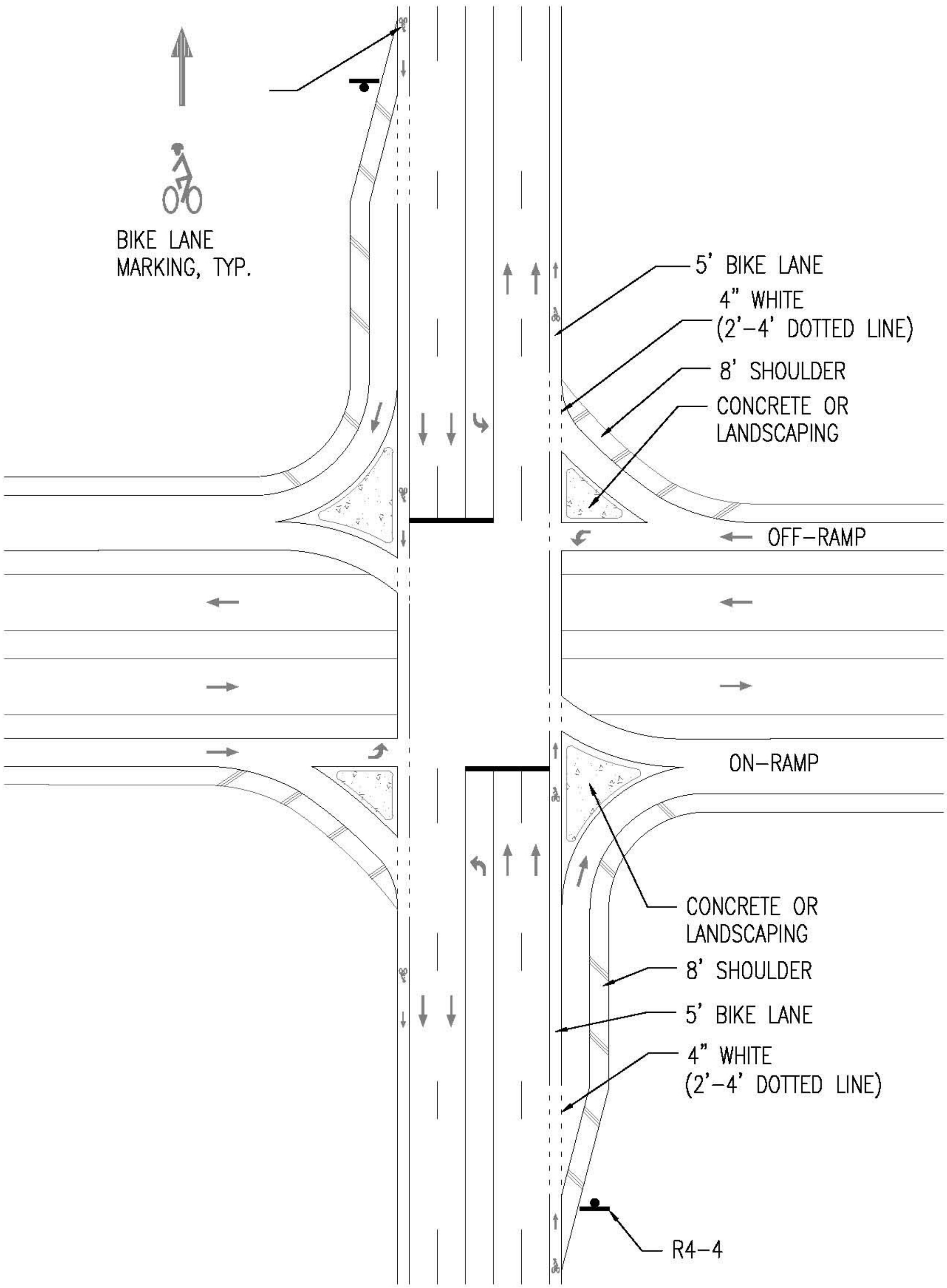


Figure 14-18 Typical Bike Lane- Compact Interchange

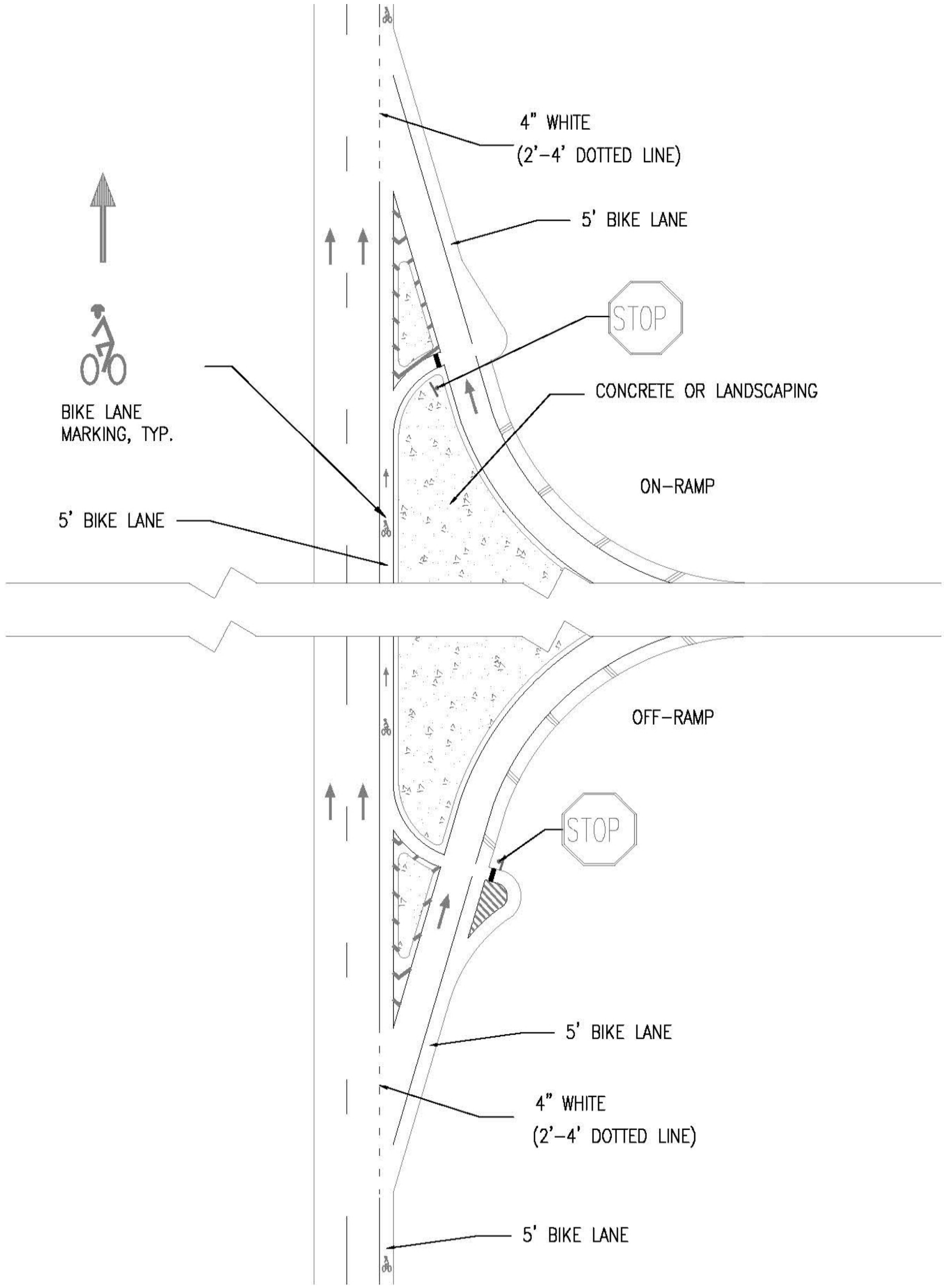


Figure 14-19 Typical Bike Lane-Rural Interchange

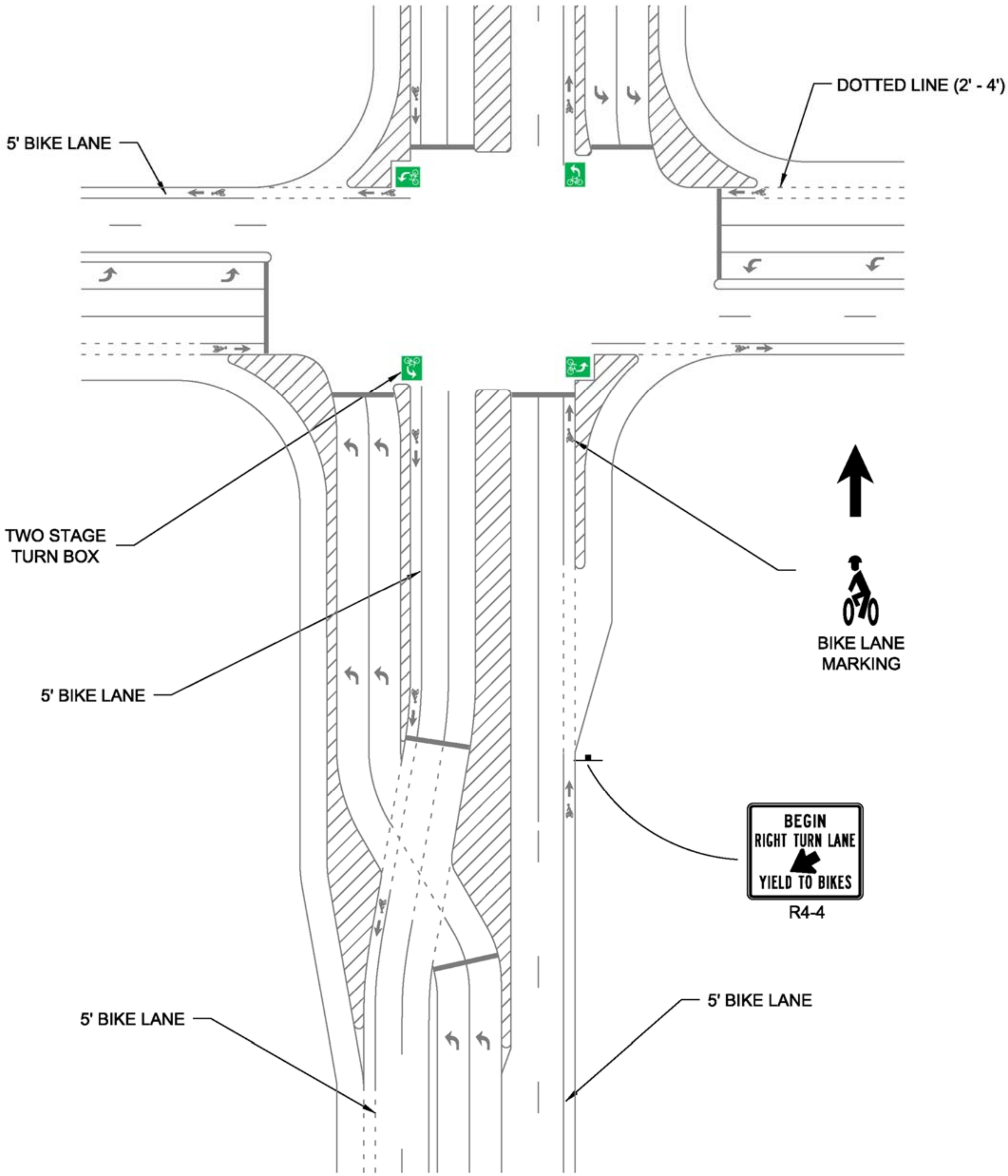


Figure 14-20 Typical Bike Lane-Continuous Flow Intersection

14.1.6.4.2 Two-Stage Turn Queuing Box

At some intersections, making a left turn by merging across traffic to a left turn lane, may be inconvenient, uncomfortable, or unsafe for bicyclists. The Colorado Revised Statutes (Section 42-4-1412(8)(a)) allows a bicyclist to turn left by merging to a left turn lane and turning just as any other vehicle, or by making a two-stage left turn as follows:

A person riding a bicycle or electrical assisted bicycle intending to turn left shall approach the turn as closely as practicable to the right-hand curb or edge of the roadway. After proceeding across the intersecting roadway to the far corner of the curb or intersection of the roadway edges, the bicyclist shall stop, as much as practicable, out of the way of traffic. After stopping, the bicyclist shall yield to any traffic proceeding in either direction along the roadway that the bicyclist had been using. After yielding and complying with any official traffic control device or police officer regulating traffic on the highway along which the bicyclist intends *to proceed, the bicyclist may proceed in the new direction*.[1]

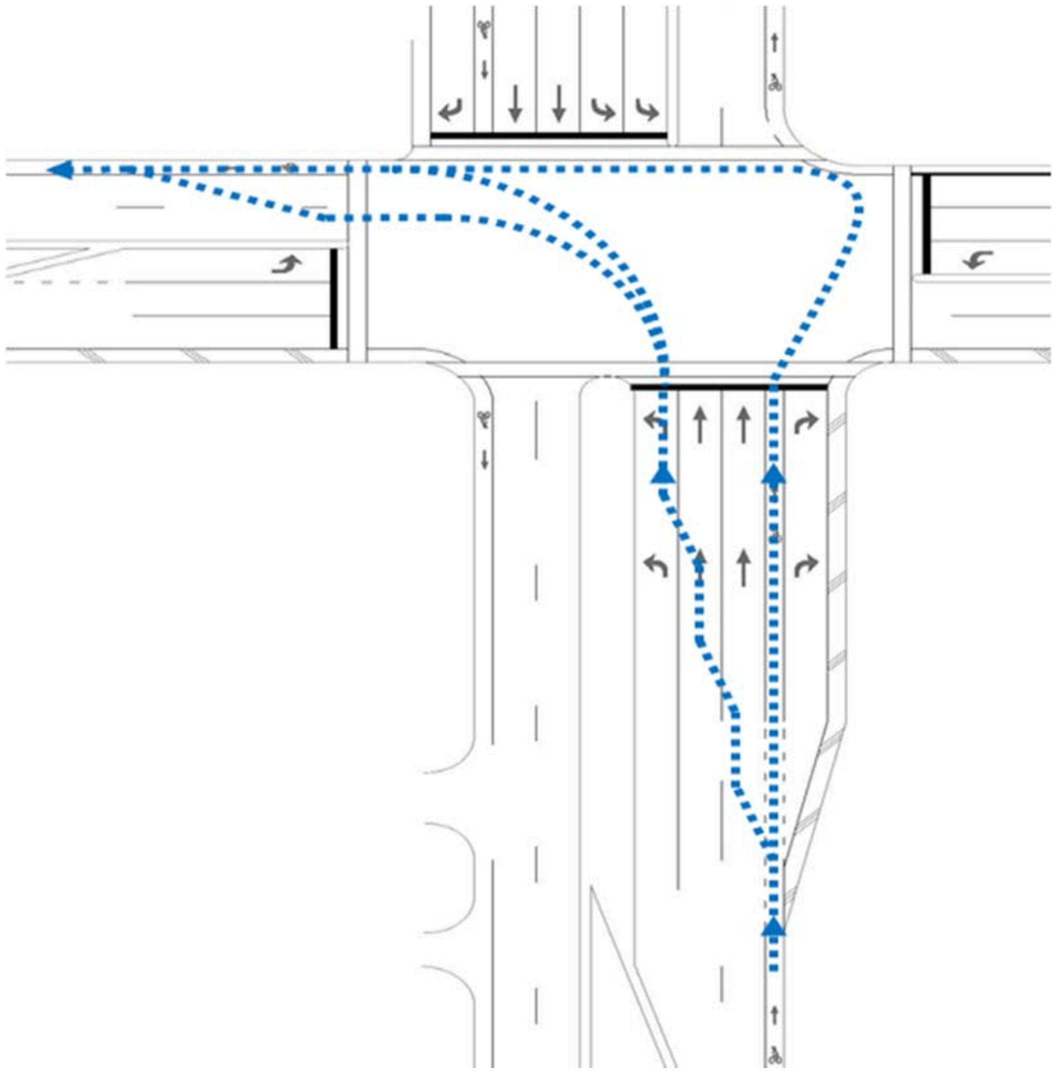


Figure 14-21 Common Maneuvers for Bicyclists Turning Left at an Intersection

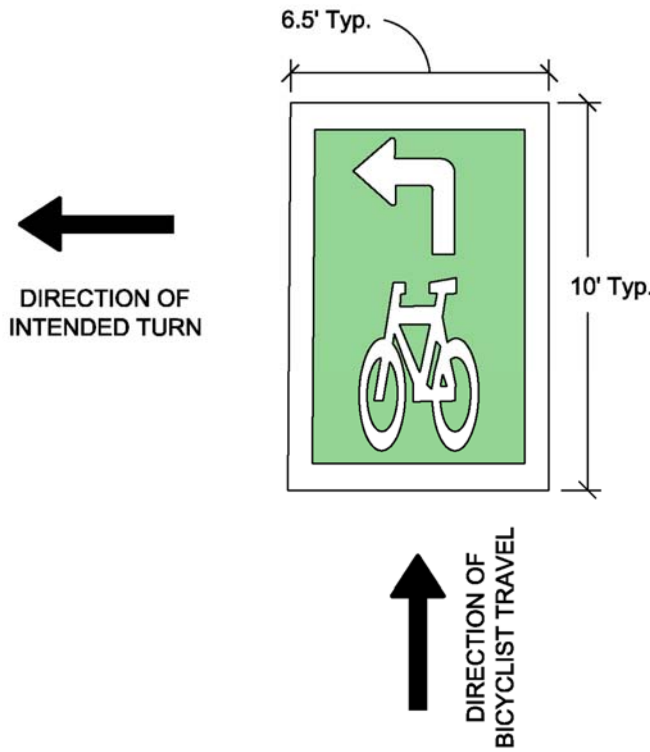


Figure 14-22 Two-Stage Left Turn Box

A two-stage turn queuing box is a designated area at an intersection intended to provide bicyclists a place to wait before proceeding in a different direction of travel. It facilitates the two-stage turn described in the statutes. A two-stage turn queuing box should be located outside of the path of turning traffic so that it does not conflict with the right turn on red movement. A NO TURN ON RED (R10-11) sign shall be installed where a two-stage turn queuing box is not located outside the path of right turning traffic. A two-stage turn queuing box should be located downstream of the crosswalk and stop line. A bicycle symbol should be placed in the two-stage turn queuing box oriented in the direction in which the bicyclists enter the box, along with an arrow showing the direction of turn, (Figure 14-22).

Passive detection of bicycles in the two-stage turn queuing box should be provided if detection is required to actuate the signal which allows bicyclists to cross. A two-stage turn queuing box is most commonly used for left turns, but it may be used for right turns from the left side of a one-way roadway. Green colored pavement may be used within the two-stage turn queuing box.

Two-stage bike boxes at an intersection are shown in Figure 14-23.

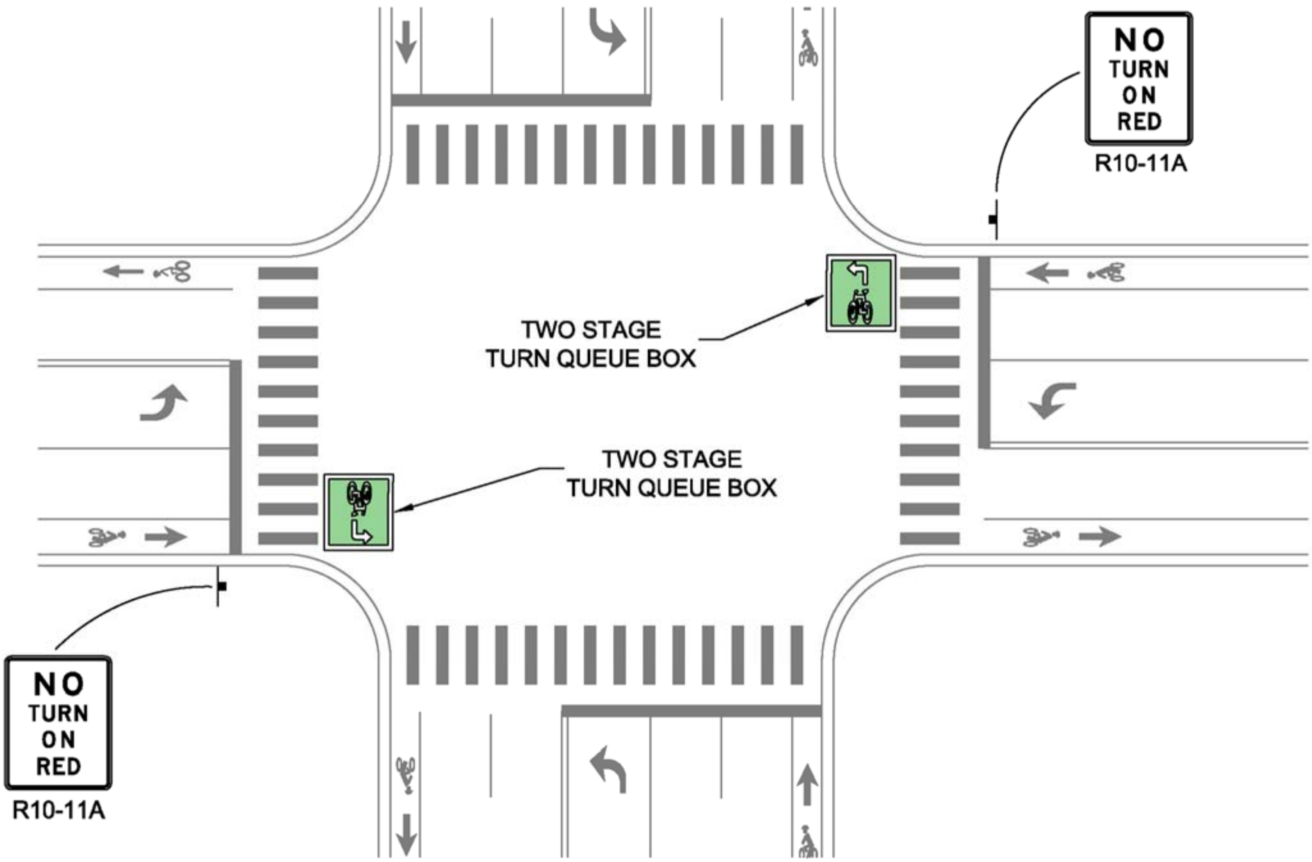


Figure 14-23 Example of Two-Stage Turn Queue Box at an Intersections.

14.1.6.5 Buffered Bike Lanes

A buffered bicycle lane is a bicycle lane that is separated from the adjacent general-purpose lane or parking lane by a pattern of standard longitudinal markings. Buffered bike lanes appeal to a wider cross-section of bicyclists and: provide greater shy distance between traffic and bicyclists, reduce the possibility of a wide bicycle lane being misconstrued as a travel or parking lane, and delineate a space between a parking lane and an adjacent bicycle lane.

The buffer markings consists of two longitudinal white lines and may incorporate an interior diagonal cross hatch or chevron (Figure 14-24). These transverse markings shall be included when the buffer space is greater than 3 feet in width. The minimum buffer width should be no less than 18 inches. The spacing for transverse markings will vary based upon the speed of the adjacent roadway, on higher speed roadways less frequent hatching may be needed. The width of the buffer will vary depending upon such conditions as motor vehicle speed, percentage of heavy vehicles, roadway cross slopes, and desired level of accommodations of bicycles. Guidelines for buffered preferential lanes can be found in the MUTCD in section 3D-01. The FHWA *Separated Bike Lane Planning and Design Guide* and the National Association of City Transportaton

Officials (NACTO) *Urban Bikeway Design Guide* also offers further design guidance for buffered bicycle lanes (60)(61).

Buffered bicycle lanes may be considered anywhere a standard bicycle lane is being considered, and may be given special consideration for roadways that exhibit high volumes or travel speeds. In some locations it may be desirable to use less than the full space available for a bike lane. Such locations include sections of roadway where a wide bike lane might be perceived as on-street parking or another travel lane. In these locations a buffered bike lane may be considered. A buffered bike lane may be considered where a bike lane of six or more feet is being provided to meet a minimum level of accommodation.

A buffer can also be provided between a parking lane and a bike lane to reduce the potential for a bicyclists to ride in a parked cars door swing zone. A buffer area provides a greater separation between the bicycle lane and adjacent lanes than is provided by a single normal or wide lane line.

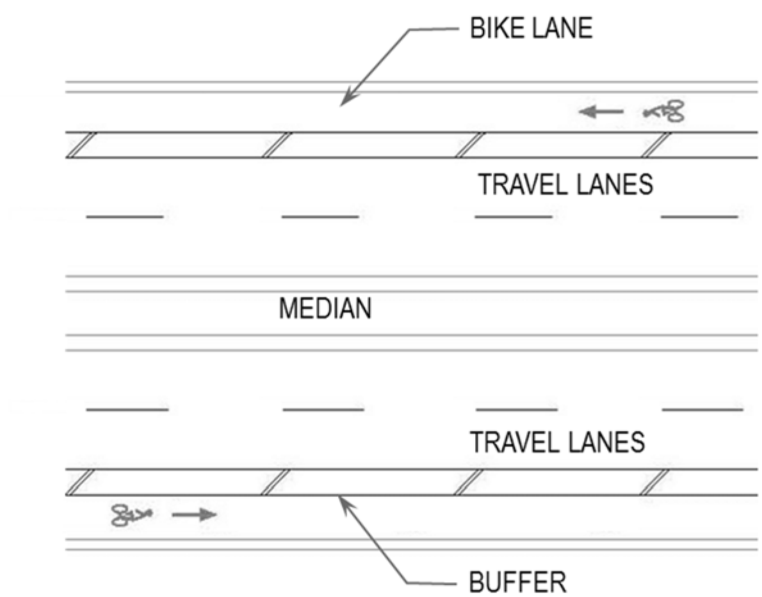


Figure 14-24 Buffered Bike Lane

14.1.6.5.1 Buffered Bike Lanes at Intersections

Buffered bike lanes should be striped much as non-buffered bike lanes at intersections.

As described in Section 14.1.6.4 Bike Lanes at Driveways and Intersections, prior to intersections, the bike lane marking is discontinued or dotted to support the legal requirements for turning motorists and to help inform the bicyclists that they are entering a potential conflict area. At intersections where a dotted bike lane line would be used, consideration should be given to terminating the buffer between the bike lane and the general travel lanes. Figure 14-26 illustrates a buffered bike lane being used at an intersection where the buffer and bike lane width becomes a right turn lane.

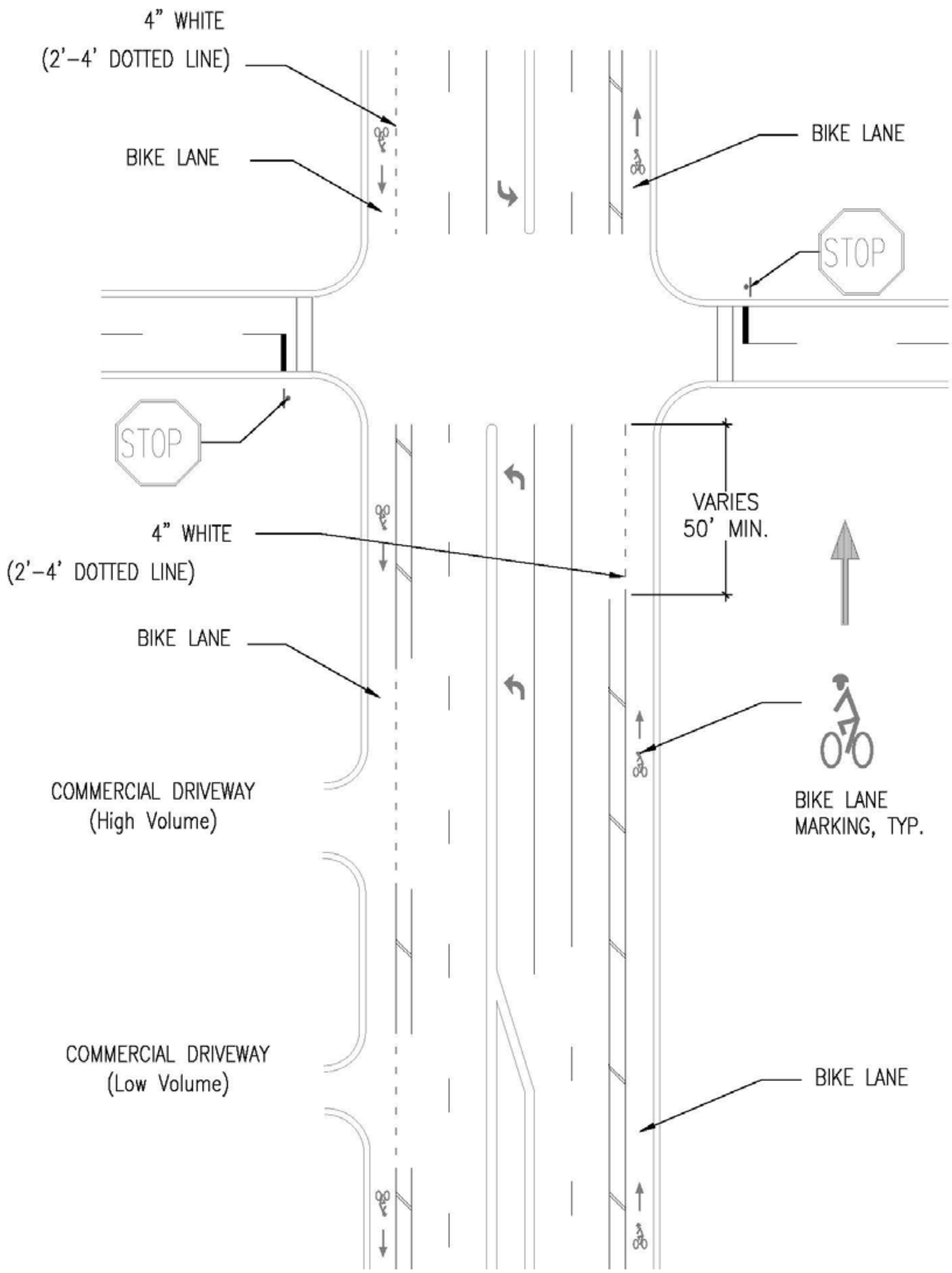


Figure 14-25 Detail of Typical Buffered Bike Lane Designation

At locations where it is desirable to include a right turn lane, but there is not adequate cross section width to provide bike lanes and a right turn lane, SHARED LANE MARKINGS can be used to guide bicyclists to the left side of a designated right turn lane. This option should only be used where there is a receiving bike lane or shoulder on the far side of the intersection.

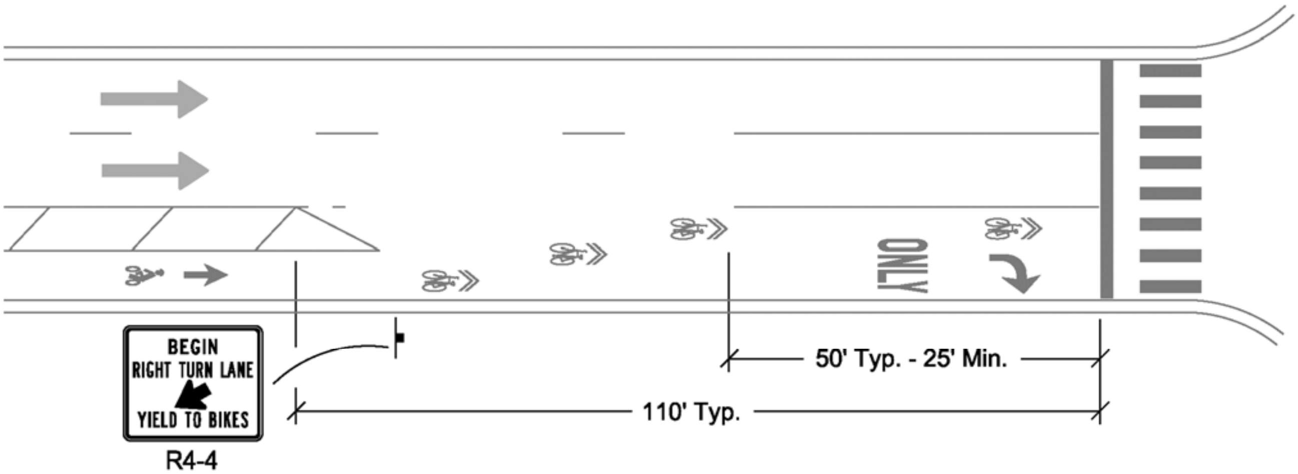


Figure 14-26 Sample Buffered Bike Lane Transition at Intersection with Right Turn Lane

14.1.7 Detection of Bicycles at Signalized Intersections

Various detection technologies can be used to detect bicyclist at intersections. The most common in Colorado are video detection and loop detection. Video detection is effective if cyclists are using the travel lanes for which detection is provided. This may exclude right turn lanes but should include left turn lanes.

There is a perception among many cyclists and roadway engineers that inductive loops do not detect the presence of bicycles, this perception results from bicyclists not waiting in an optimal spot for detection. Research has shown that inductive loops are highly reliable at detecting steel and aluminum bicycles when bicycles are in the proper position **(24).**

Calibrating loop sensitivity to detect bicycles is a principal challenge of signal hardware design, this has led to development of numerous loop configuration solutions. The 6-foot by 40-foot quadripole loops shown on standard drawing S-614-43 Traffic Loop and Miscellaneous Signal Details should be capable of detecting bicycles.

There are two basic strategies to improve detection of bicycles: to direct bicyclists to the area of optimal loop sensitivity and alternatively to place new loops in spots where cyclists are likely to be waiting, such as in the bike lane or at the right edge of the pavement. It is recommended that these strategies for optimizing loop detection of bicyclists be employed before investigating a substantial investment of new technology; the technology already in place at many intersections is likely quite capable of detecting bicyclists.

One of the simplest ways to facilitate the detection of bicyclists at traffic signals is to mark the spot on the roadway where a given loop will detect a bicycle. The *MUTCD* provides for a symbol that may be placed on the pavement to indicate the optimum position for a bicyclist to actuate the signal **(25)**. Used in conjunction with the BICYCLE SIGNAL ACTUATION sign (R10-22)

(26) (see Figure 14-27), this symbol can eliminate the problem of bicycle detection for any intersection movement where the loops can detect bicyclists.

New loops should be of a type that will detect bicycles.

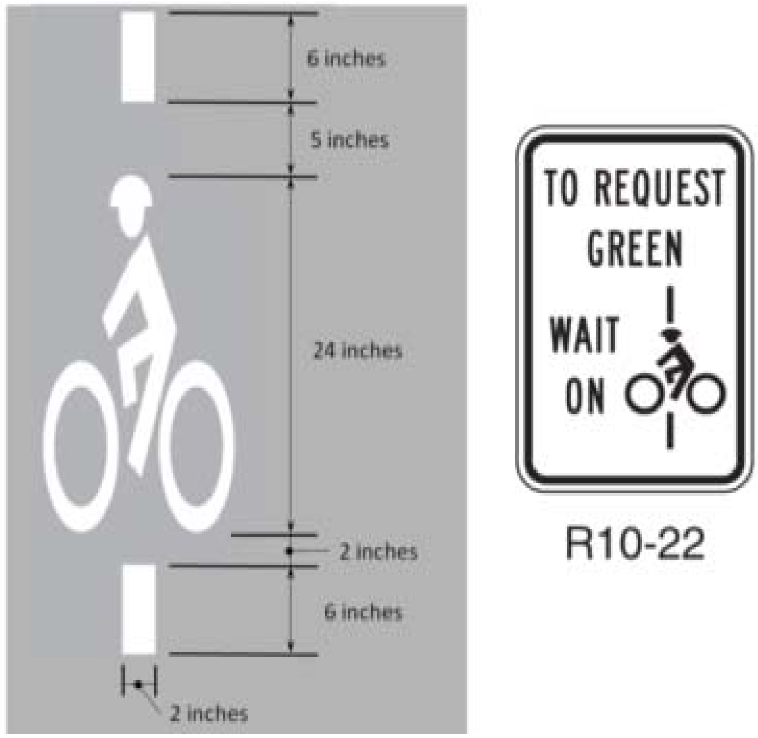


Figure 14-27 Bike Detection Symbol and Bicycle Signal Actuation Sign

14.1.7.1 Signal Detection Loops in Bike Lanes

Changing lanes at an intersection to cause a signal change is not normal vehicular behavior, yet bicyclists are frequently required to do so. In the interest of providing consistent treatments between modes, bike lane detection should be considered at locations where signal change is unlikely without detection.

The recommended loop type for bike lanes is a quadripole loop of reduced size (2-foot x 10-foot). These loops are highly sensitive to objects in the area immediately above them, but detection falls off rapidly outside of this sensitivity field; this means that cars in adjacent lanes will not be detected.

14.1.7.2 Signal Timing for Bicycles

The MUTCD requires that signal timing and actuation on bikeways be reviewed and adjusted to consider the needs of bicyclists **(27)**. Meeting the needs of bicyclists on bikeways means providing adequate minimum green times and adequate change periods.

The minimum green time allows bicyclists to start from a stopped condition, cross, and clear the intersection. For the crossing of narrow roadways, the bicyclists may not accelerate to full speed before clearing the intersection. On wider roads, the bicyclist will accelerate to full speed and may require additional time to finish crossing and clear the intersections. The equations to calculate minimum green time are as follows:

$G_{min} = 1.0 + 1.15\sqrt{W+6}$ Where W ≤ 72 feet

$G_{min} = 10.8 + \frac{(W-72)}{14.7}$ Where W > 72 feet

and

G_{min} = minimum green time (sec)

W = width of intersection (ft)

Typically the minimum change period is calculated using the following equation **(28):**

$$CP = \left[t + \frac{1.47v}{2(a+32.2g)}\right] + \left[\frac{W+L_v}{1.47v}\right]$$

where:

CP = change period (yellow change plus red clearance intervals),(sec)

t = perception-reaction time to the onset of a yellow indication, s, assume 1 (sec)

v = approach speed (mph)(assume 10 MPH for a bicycle)

a = deceleration rate in response to the onset of a yellow indication, (ft/sec),(assume 5 ft/sec for a bicycle)

g = grade, with uphill positive and downhill negative (percent grade / 100),(ft/ft)

W = width of intersection (ft)

L_v = length of vehicle, (ft)(assume 6 ft for bicycle)

At wide intersections, the clearance interval provided for motorists may not be long enough to provide for bicyclists to clear the intersection. Advance loops in bike lanes or on shoulders can provide an extended green time to allow bicyclists to clear the intersection before the conflicting traffic gets a green signal. Alternatively, a supplemental bicycle specific signal (see Section 14.2.16.6.3 Bicycle Signals) with a supplement plaque stating BICYCLE SIGNAL could be provided for bicyclists.

At installations where visibility-limited signal faces are used, signal faces shall be adjusted so bicyclists for whom the indications are intended can see the signal indications. If the visibility-limited signal indications cannot be aimed to serve the bicyclist, then separate signal indications shall be provided for the bicyclist.

14.1.8 Bike Lanes at Roundabouts

Bike lanes are not carried through roundabouts. The MUTCD states that bike lane markings should stop at least 100 feet prior to the approach of a roundabout. Following the end of a bike lane, a pathway must be provided for bicyclists to exit the roadway, if they choose. A SHARED LANE MARKING may be used through the roundabout. Figure 14-28 is an example of a multi-lane roundabout.

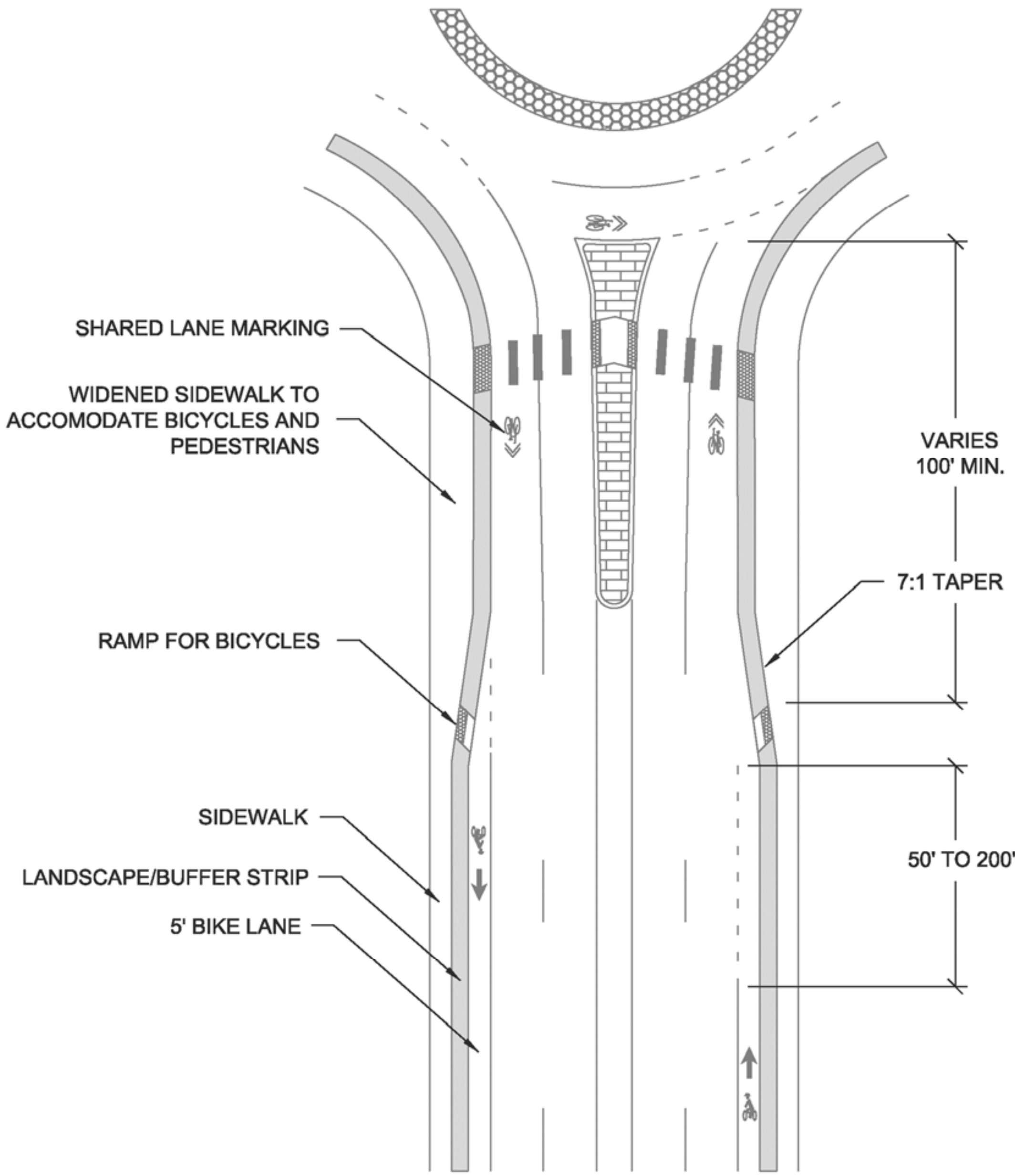


Figure 14-28 Multi-lane Roundabout

14.1.9 Separated Bike Lanes (Cycle track)

Separated bike lanes are bicycle lanes which are separate from general travel lanes and the sidewalk. They are not the same as shared use paths because they are bicycle-only facilities. They are distinct from buffered bike lanes because there is a physical separation, such as a raised island or parked cars, between the bicyclists and the outside travel lane. Operationally, they can be very challenging, particularly at intersections with driveways and streets.

For guidance on the design of cycle tracks refer to the FHWA document *Separated Bike Lane Planning and Design Guide* and the NACTO *Urban Bikeway Design Guide*.

14.1.10 Bicycle Boulevards

A bicycle boulevard is a local street or series of contiguous street segments that have been modified to provide enhanced accommodation for bicyclists while discouraging through automobile travel. Local motor vehicle access is maintained along the streets. Bicycle boulevards would not be implemented on CDOT roadways. However, they may be used to improve alternative routes (see Section 14.1.10).

Bicycle boulevards often make use of low volume, very low speed local streets. SHARED LANE MARKINGS may be used along bike boulevards. Often bicycle boulevards include bicycle friendly traffic calming treatments (speed cushions, mini traffic circles) to reduce speeds of motor vehicles along the roadway. Some portions of a bike boulevard may be on busier roads with bike lanes. Through motor vehicle traffic can be discouraged using traffic diverters at intersections. Bicycle boulevards can be created by connecting the ends of cul de sac roadways with bikeways. At intersections the bicycle boulevard should be given priority over side streets. Additionally, since bike boulevards typically serve as bike routes, wayfinding signage should be provided.

One potential obstacle to implementing bike boulevards is the crossing of major roadways. Improvements to signal timing and detection or the provision of enhanced crossing treatments (activated beacons, raised medians) where no signals exist will make a bicycle boulevard more appealing to cyclists.

Another challenge related to bike boulevards can be frequent opposition voiced by those who live along the streets being altered. Other motorists who travel on the street may feel the same way because of altered travel patterns for the auto mode. Designers considering the implementation of a bike boulevard should be aware of these considerations and should accordingly plan for early and sustained public outreach to the project’s neighbors, communities and municipalities.

14.1.11 Alternative Routes

On some projects it may not be possible to improve the roadway to accommodate bicyclists. In these cases it may be possible to improve an adjacent street to provide an alternative route for bicyclists to access destinations that would be served by the primary project roadway. Alternative routes could potentially be improved using some of the treatments described in this chapter.

In addition to the accommodations provided along the alternative route, several other factors must be addressed when considering whether or not an alternative route provides a suitable accommodation for bicyclists:

- Geometric delay - This is the delay caused to the bicyclists by increased distance they must travel to use the alternative route. If an alternative route significantly increases the distance and time a bicyclist must travel to access a destination it will be less likely to be used.
- Control delay - This is the delay caused by the increasing the number of STOP signs or red traffic signals along a route. Often the primary corridor is given the majority of the green time at signals and does not often have to stop at minor street intersections. If the alternative route is a local street that must stop at every cross street and gets minimal green time at signalized intersections, bicyclists will be less likely to use it.
- Access to destinations - An alternative route must provide access to the trip destinations along the primary corridor or it will not be a practical option for bicyclists.
- Safety - Any alternative route being considered for improvement should be subject to a safety assessment. This would include reviewing crashes along the route as well as identifying potential safety concerns associated with accessing the primary project corridor from the alternative route.

14.1.12 Other Roadway Considerations

14.1.12.1 Roadway Cross Slope

The typical cross slopes provided for roadways will usually accommodate cyclists. Cross slopes of 5% or less are desirable for bicycles. However, the AASHTO *Guide for the Development of Bicycle Facilities* allows superelevation rates up to 8%.

14.1.12.2 Drainage Inlets and Utility Covers

Placement of drainage inlet grates should be avoided within a bicycle facility regardless of whether that facility is a bike lane, shoulder, or shared lane. If this is not possible, drainage inlet grates should be bicycle-safe. Utility covers and drainage grates should be installed to be flush with the pavement. The construction of new roadway facilities should consider the use of curb inlets as opposed to gutter pan drop inlets.

Drainage inlet grates with slots or gaps parallel to the roadway can trap a bicycle’s front wheel and seriously damage the bicycle or injure the cyclist. These types of grates should be replaced with bicycle-safe grates that maintain the required hydraulic capacity for the inlet (Figure 14-29). A bicycle-safe grate should have, at a minimum, bars perpendicular to the travel direction at a 4 inch center-to-center spacing

For safety considerations, any utility cover or drainage inlet located on a bicycle facility that has a gap or opening parallel to the roadway should be replaced or corrected as soon as possible. If a drop inlet with parallel slots cannot be replaced, an obstruction marking should be placed on the pavement prior to the inlet (Figure 14-30).

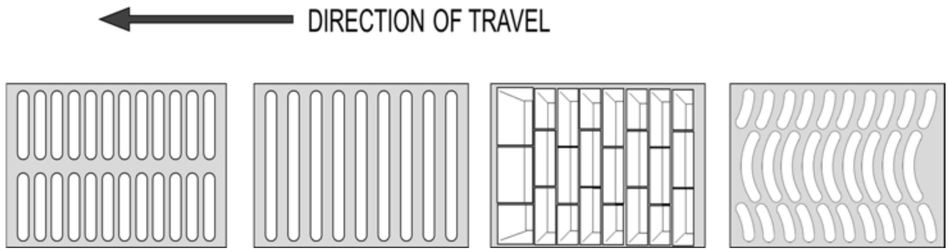


Figure 14-29 Bicycle Compatible Drainage Grates

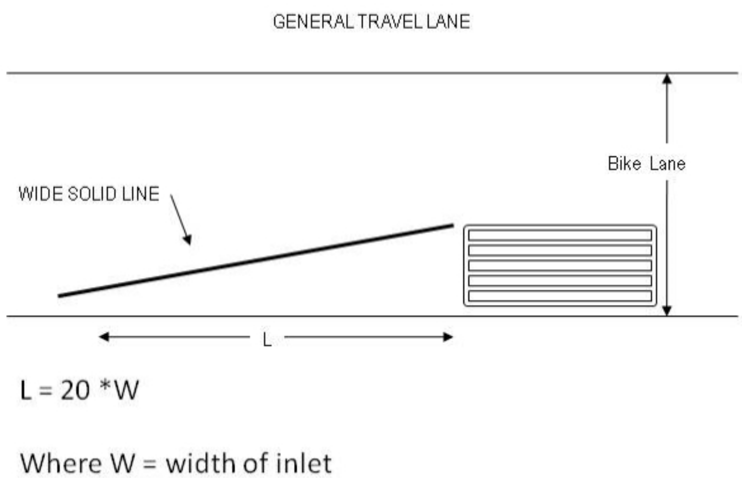


Figure 14-30 Bicycle Obstruction Marking in Advance of a Drop Inlet

14.1.12.3 Railroad Crossings

Ideally travel ways should cross rail lines at right angles. The more the railroad crossing deviates from a right angle, the greater the potential for a cyclist's front wheel to be trapped in the tracks, causing the loss of steering control and a crash. SKEWED CROSSING warning signs (W10-12) should be considered for the approach to the crossing.

A special treatment should be considered for railroad crossings with angles less than 45 degrees. It is recommended that a special path be provided for bicyclists to cross the tracks at a right angle. The simplest approach would be to provide a pavement widening at the crossing. Figure 14-31 shows two scenarios of potential skewed crossing treatments. Additionally, pavement markings can be provided to direct bicyclists to the preferred path of travel.

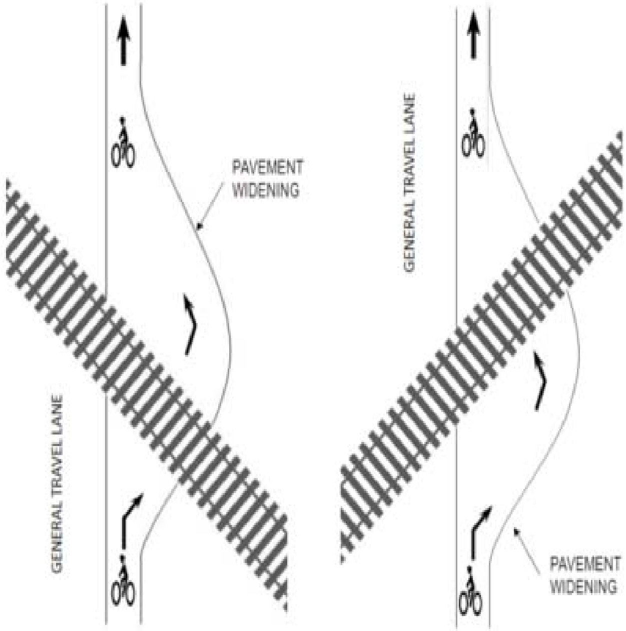


Figure 14-31 Potential Treatments at a Skewed Railroad Crossing

14.1.12.4 Bridges and Tunnels

The FHWA Design Guidance and Policy Statement **(29)** states: "A bridge that is likely to remain in place for 50 years should be built with sufficient width for safe bicycle and pedestrian use (sidewalks and shoulders) in anticipation that facilities will be available at either end of the bridge even if that is not currently the case. Design bridges with sidewalks and shoulders or bike lanes on both sides of the structure." Tunnels should also be designed to accommodate bicyclists and pedestrians.

14.2 SHARED USE PATHS

Shared use paths are physically separated from motorized vehicular traffic by either a physical barrier or clear space. They are often on their own alignments but may be located within the right-of-way of an adjacent roadway.

Since shared use paths are intended for use by many modes (such as pedestrians, persons with disabilities, etc.) they must be made ADA compliant to the maximum extent feasible (see Section 14.3).

14.2.1 Surface Treatments

14.2.1.1 Paved Shared Use Path

Most CDOT shared use path projects will be paved. Asphalt and Portland cement concrete are the two most common surfaces for shared use paths. For rigid pavement design information, refer to the CDOT *Pavement Design Manual.* The Materials Engineer should be consulted for flexible pavement design information. On Portland cement concrete pavements, the transverse joints should be saw cut, rather than tooled, to provide for a smoother ride. Skid resistance should not be reduced, broom finish or burlap drag surfaces should be provided.

Where paved shared use paths cross unpaved roadways or driveways, the road or drive should be paved 20 feet on each side of the shared use path to help minimize debris accumulation on the path.

14.2.1.2 Unpaved Shared Use Paths

In areas where path use is expected to be primarily recreational, unpaved surfaces may be acceptable for shared use paths. Materials should be chosen to ensure the ADA requirements for a firm, stable, slip resistant surface are met. Even when meeting ADA criteria, some users such as in-line skaters, kick scooters, and skateboarders may be unable to use unpaved shared use paths.

On unpaved shared use paths, grades of greater than 3 percent may result in erosion problems and bicycle handling problems for some bicyclists. Additionally, snow plowing may be impractical on unpaved shared use paths.

14.2.2 Design Speed

As with roadways, the design speed selected for shared use paths dictates other design criteria (sight distance, curve alignments). Consequently, the selection of an appropriate design speed is important to maximize the flexibility of design when developing a shared use path.

Design speeds range from 12 to 30 mph. Two mph increments of design speed should be used for less than 20 mph, and 5 mph increments should be used above 20 mph.

An 18 mph design speed is generally sufficient for most paths in relatively flat areas (generally less than 2 percent grades). If it is expected that there will be significant use by recumbent bicyclists, the minimum design speed should be to 18 mph (7).

Design speeds lower than 18 mph may be used in areas where the expected riding population is anticipated to be made up of lower speed users such as children. A design speed of less than 14 mph should be used only in unusual circumstances. Justification based upon environmental context and user types should be provided when using a design speed less than 14 mph.

Lower design speeds may be used on the approach to roadway crossing points or hazards. Traffic control and geometric features should be used together to reduce speeds in these locations (see Section 14.2.10.6).

Where sustained grades exceeding 4 percent in excess of 300 feet in length are required, an increased design speed should be used. They should be based upon the anticipated travel speeds of cyclists traveling downhill. Thirty mph should be the maximum design speed used in all but the most unusual cases.

14.2.3 Sight Distance

As stated in Chapter 3 of this Roadway Design Guide, a critical element in assuring safe and efficient operation of a vehicle on a highway is the ability to see ahead. Sight distance is the distance along a roadway or path throughout which an object of specified height is continuously visible to a bicyclist. In a vertical plane, this distance is dependent on the height of the bicyclist's eye above the road or path surface, the specified object height above the road surface, and the height and lateral position of obstructions such as cut slopes, guardrail, and retaining walls within the bicyclists' line of sight. Horizontal alignment, including the routing of a path around visual screens, can also impact sight distance and should be considered. Sight distance of sufficient length must be provided to allow a bicyclist to avoid striking unexpected objects in the traveled way.

14.2.3.1 Stopping Sight Distance

Stopping sight distance is the sum of two distances:

- The distance a bicycle travels from the instant the bicyclist sights an object necessitating a stop to the instant the brakes are applied (brake reaction distance), and
- The distance required to stop the bicycle from the instant brake application begins (braking distance).

Stopping sight distance is measured from the bicyclist's eyes, which are assumed to be 4.5 feet above the pavement, to an object flush with the surface of the shared use path. If it is found that a significant number of recumbent cyclists are represented in the local cycling population, an eye height of 2.8 feet should be used **(7)**. Distances greater than the minimum stopping sight distance provide an additional measure of safety and should be considered where practical.

On downhill grades, gravity acts against braking forces and increases the distance required to stop. On uphill grades gravity reduces the distance required to stop. The effect of grades is represented in stopping sight distance values.

The equation for stopping sight distance, assuming a 2.5 second reaction time, is

$$S = 3.67V + \frac{V^2}{30(f + G)}$$

Where,

S = stopping sight distance (ft)

V = design speed (mph)

f = friction factor (assume 0.16 for a typical bicycle)

G = grade in (ft/ft)

Table 14-4 shows sight distances for level roadways and roadways with grade for various design speeds. See also Chapter 3 for adjustments for grades.

Design Speed (mph)	Stopping Sight Distance (Design Values)						
	No grade adjustment	% Down Grade			% Up Grade		
		3	6	9	3	6	9
8	43	46	51	60	41	40	38
10	58	63	71	85	55	52	51
12	75	81	93	113	70	66	64
14	93	102	117	145	86	82	78
16	113	125	145	181	104	98	93
18	134	150	175	221	123	116	110
20	157	176	207	264	144	135	127
25	222	253	301	390	202	187	176
30	298	341	411	539	268	247	231

Table 14-4 Stopping Sight Distance for Bicycles

14.2.3.2 Sight Distance on Horizontal Curves

Sight distance on horizontal curves on shared use paths may be obtained with the aid of Figure 14-32 and Table 14-5. The line of sight is assumed to intercept the obstruction at the midpoint of the sight line and at the surface of the center of the inside lane. The middle horizontal sightline offset (HSO) is obtained from the equation in Figure 14-32 and from Table 14-5.

The stopping sight distance in Table 14-5 is the stopping sight distance determined using the equation or table from Section 14.2.3.1. The minimum radii for horizontal curves are addressed in Section 14.2.7

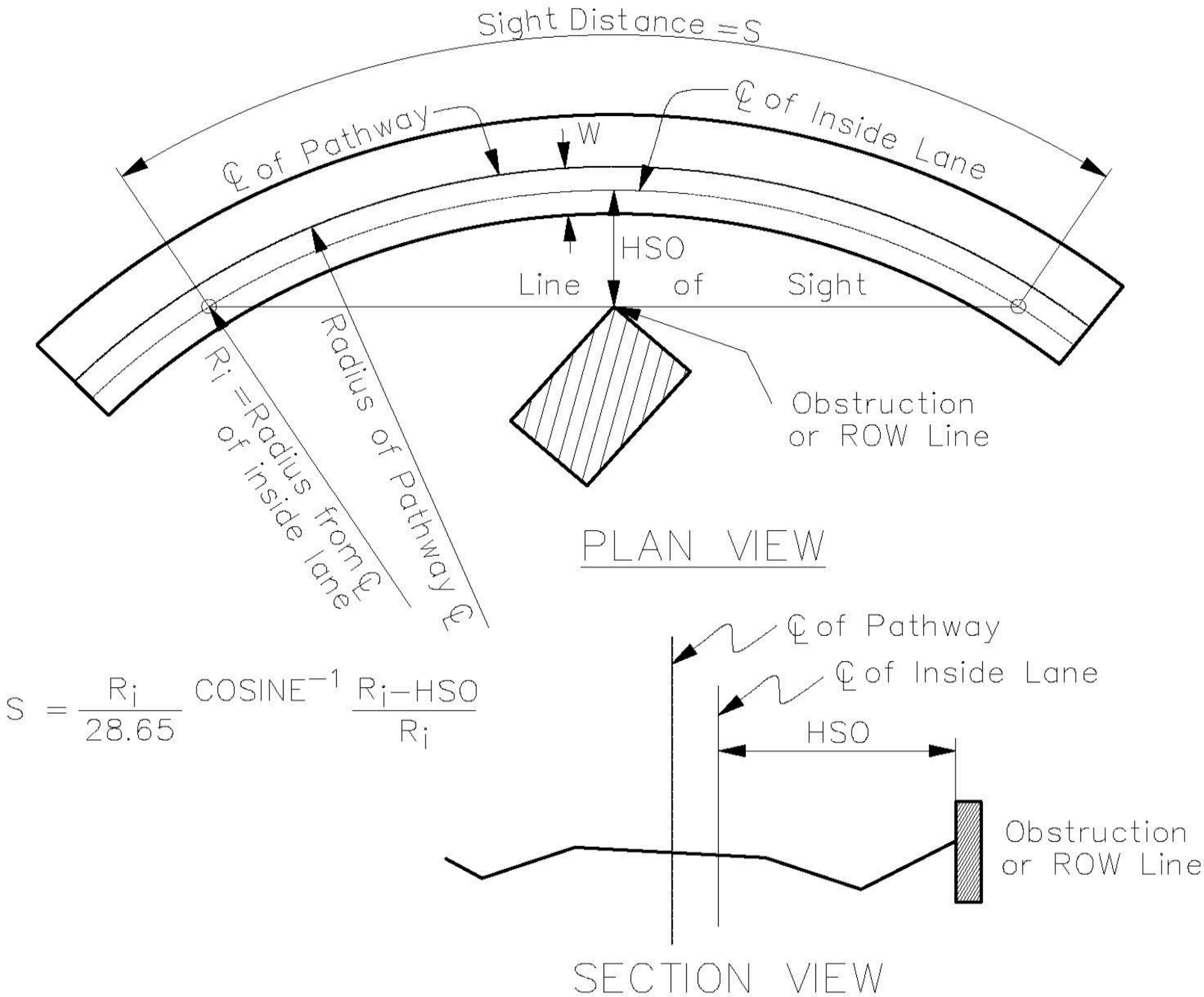


Figure 14-32 Stopping Sight Distance on a Shared Use Path Horizontal Curve

	Stopping Sight Distance												
Curve Radius (ft)	20	40	60	80	100	125	150	175	200	225	250	275	300
15	3.2	11.5	21.2	28.3	29.7	22.8	10.7	1.5	1.1	9.8	21.9	29.5	27.6
20	2.4	9.2	18.6	28.3	36.0	40.0	36.4	26.6	14.3	4.2	0.0	3.4	13.1
25	2.0	7.6	15.9	25.7	35.4	45.0	49.8	48.4	41.3	30.3	17.9	7.3	1.0
35	1.4	5.6	12.1	20.5	30.0	42.5	54.0	63.0	68.6	69.9	66.8	59.7	49.5
50	1.0	3.9	8.7	15.2	23.0	34.2	46.5	58.9	70.8	81.4	90.1	96.2	99.5
75	0.7	2.7	5.9	10.4	16.1	24.6	34.5	45.5	57.4	69.7	82.2	94.5	106.2
100	0.5	2.0	4.5	7.9	12.2	18.9	26.8	35.9	46.0	56.9	68.5	80.6	92.9
125	0.4	1.6	3.6	6.3	9.9	15.3	21.8	29.4	37.9	47.3	57.5	68.3	79.7
150	0.3	1.3	3.0	5.3	8.3	12.8	18.4	24.8	32.1	40.3	49.1	58.7	69.0
175	0.3	1.1	2.6	4.6	7.1	11.0	15.8	21.4	27.8	34.9	42.8	51.3	60.5
200	0.2	1.0	2.2	4.0	6.2	9.7	13.9	18.8	24.5	30.8	37.8	45.4	53.7
225	0.2	0.9	2.0	3.5	5.5	8.6	12.4	16.8	21.9	27.5	33.8	40.7	48.2
250	0.2	0.8	1.8	3.2	5.0	7.8	11.2	15.2	19.7	24.9	30.6	36.9	43.7
300	0.2	0.7	1.5	2.7	4.2	6.5	9.3	12.7	16.5	20.9	25.7	31.0	36.7
350	0.1	0.6	1.3	2.3	3.6	5.6	8.0	10.9	14.2	17.9	22.1	26.7	31.7
400	0.1	0.5	1.1	2.0	3.1	4.9	7.0	9.5	12.4	15.7	19.4	23.4	27.8
450	0.1	0.4	1.0	1.8	2.8	4.3	6.2	8.5	11.1	14.0	17.3	20.8	24.8
500	0.1	0.4	0.9	1.6	2.5	3.9	5.6	7.6	10.0	12.6	15.5	18.8	22.3
600	0.1	0.3	0.7	1.3	2.1	3.3	4.7	6.4	8.3	10.5	13.0	15.7	18.7
700	0.1	0.3	0.6	1.1	1.8	2.8	4.0	5.5	7.1	9.0	11.1	13.5	16.0
800	0.1	0.3	0.6	1.0	1.6	2.4	3.5	4.8	6.2	7.9	9.7	11.8	14.0
900	0.1	0.2	0.5	0.9	1.4	2.2	3.1	4.3	5.6	7.0	8.7	10.5	12.5
1000	0.1	0.2	0.5	0.8	1.2	2.0	2.8	3.8	5.0	6.3	7.8	9.4	11.2

Table 14-5 Minimum Horizontal Clearance for Horizontal Sightline Offset for Horizontal Curves

14.2.3.3 Sight Distance on Vertical Curves

Sight distance on vertical curves is required to allow bicyclists to see objects on the path over the crest of vertical curves or obstacles that are located beyond overhanging visual obstructions on sag vertical curves. The method of calculating sight distance for bicyclists on vertical curves is essentially the same as that used for calculating the sight distance for motorists (Section 3.1.5 Sight Distance on Vertical Curves); however, the height of eye and object height need to be modified for bicycle specific values. Stopping sight distance is measured when the eye height and the height of the object are 4.5 feet (for a typical bicycle rider) and 0 feet (flush with the pavement surface) respectively.

When S is less than L,

$$S = 30\sqrt{\frac{L}{A}}$$

When S is greater than L,

$$S = \frac{L}{2} + \frac{2025}{A}$$

Where,

S = stopping sight distance (ft)

L = length of crest vertical curve (ft)

A = algebraic difference in grades (%)

Table 14-6 Minimum Length of Crest Vertical Curve Based on Stopping Sight Distance is used to select the minimum length of vertical curve necessary to provide minimum stopping sight distance at various speeds on crest vertical curves. Note that this table is for regular bicycles. For recumbent bicycles the values would need to be recalculated using equations 3-14 and 3-42 in the PGDSH)) **(1)**.

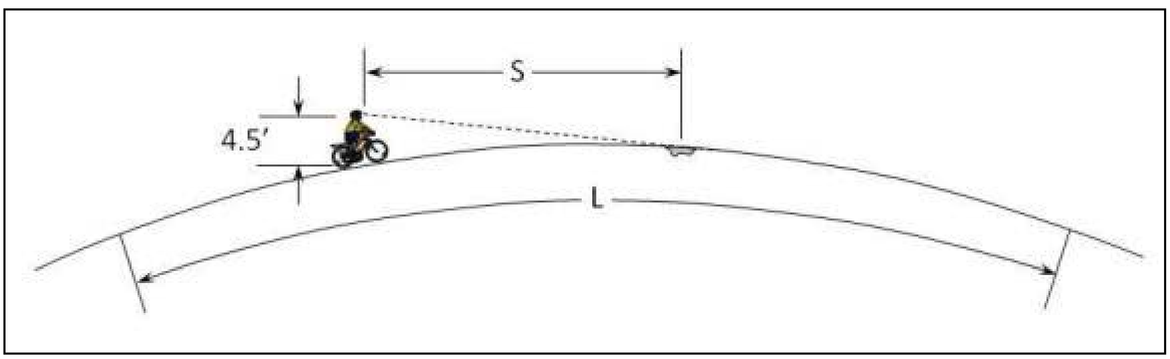


Figure 14-33 Sight Distance on Crest Vertical Curves

A (%)	S = Stopping Sight Distance (ft)														
	20	40	60	80	100	120	140	160	180	200	220	240	260	280	300
2												30	70	110	150
3								20	60	100	140	180	220	260	300
4						15	55	95	135	175	215	256	300	348	400
5					20	60	100	140	180	222	269	320	376	436	500
6				10	50	90	130	171	216	267	323	384	451	523	600
7				31	71	111	152	199	252	311	376	448	526	610	700
8			8	48	88	128	174	228	288	356	430	512	601	697	800
9			20	60	100	144	196	256	324	400	484	576	676	784	900
10			30	70	111	160	218	284	360	444	538	640	751	871	1000
11			38	78	122	176	240	313	396	489	592	704	826	958	1100
12		5	45	85	133	192	261	341	432	533	645	768	901	1045	1200
13		11	51	92	144	208	283	370	468	578	699	832	976	1132	1300
14		16	56	100	156	224	305	398	504	622	753	896	1052	1220	1400
15		20	60	107	167	240	327	427	540	667	807	960	1127	1307	1500
16		24	64	114	178	256	348	455	576	711	860	1024	1202	1394	1600
17		27	68	121	189	272	370	484	612	756	914	1088	1277	1481	1700
18		30	72	128	200	288	392	512	648	800	968	1152	1352	1568	1800
19		33	76	135	211	304	414	540	684	844	1022	1216	1427	1655	1900
20		35	80	142	222	320	436	569	720	889	1076	1280	1502	1742	2000
21		37	84	149	233	336	457	597	756	933	1129	1344	1577	1829	2100
22		39	88	156	244	352	479	626	792	978	1183	1408	1652	1916	2200
23		41	92	164	256	368	501	654	828	1022	1237	1472	1728	2004	2300
24	3	43	96	171	267	384	523	683	864	1067	1291	1536	1803	2091	2400
25	4	44	100	178	278	400	544	711	900	1111	1344	1600	1878	2178	2500

Table 14-6 Minimum Length of Crest Vertical Curve Based on Stopping Sight Distance

The primary control for designing sag vertical curves for roadways is the limitations of headlamp lighting at night. This control is reasonable for cars because they are required to have operating headlamps and headlamps are typically adjusted with a reasonable degree of consistency. While bicyclists who are riding between sunset and sunrise are required to have a headlamp, the purpose of the headlamp is to make the bicyclists visible **(30)**. There are a wide variety of headlamp designs and the light they provide for bicyclists to see the path in front of them is widely variable. Consequently, using headlamp limitations as a design control is not practical for shared use paths.

A sag curve on a shared use path must be designed so that it provides the minimum stopping sight distance described for in Section 14.2.3.1. In most cases, meeting these criteria will not be problematic. One common exception is when a path is depressed through an undercrossing, in which case the sight distances should be checked to ensure that any overhanging structure does not limit the stopping sight distance to less than that which is required.

14.2.3.4 Sight Distance at Intersections

The discussion on intersection sight distance provided in Chapter 9 of this *Roadway Design Guide* is also applicable to shared use paths. Also applicable are the procedures to determine sight distances at intersections presented in Chapter 9 of the *PGDHS*) **(1)**, using the appropriate design speed for the shared use path approaches to the intersection, for each of the cases below:

Case A --Intersections with no control (not typically used on shared use paths)

Case B -- Intersections with stop control on 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 D – Intersections with traffic signal control

Checking the sight distances for vehicles turning onto or off of the shared use path is typically not necessary. The minor roadway may be either the shared use path or the roadway.

14.2.4 Shared Use Path Width

The minimum width of pavement for a two-directional shared use path is 10 feet.

Additional width may be appropriate depending on the volume of users and mix of users on the shared use path. The FHWA has developed a level of service shared use path calculator which may be helpful in determining the appropriate width for a path based on the relative number of users expected **(31, 32)**. Pathways of up to 14 feet are recommended in locations that are anticipated to have high volumes (greater than 300 users in the peak hour), or with a high percentage of pedestrians (greater than 30 percent). An 11 foot shared use path will allow for a bicyclist to pass another traveling in the same direction at the same time a someone is approaching from the opposite direction **(31)**. Wider paths should be considered where there is expected significant use by in-line skaters, hand cyclists, adult tricyclists **(7)**, or on steep grades and through curves.

A reduced width, to as little as 8 feet, may be used only for short sections of constrained conditions and where the following conditions apply:

- Bicycle traffic is expected to be low, even on peak days or during peak hours
- Pedestrian use of the facility is not expected to be more than occasional
- Horizontal and vertical alignments provide safe and frequent passing opportunities, and
- The path will not be regularly subjected to maintenance vehicle loading conditions that would cause pavement edge damage.

In most cases it is not necessary to designate separate space for different users on shared use paths. Slower path users tend to keep right while higher speed users pass on the left. If additional encouragement is necessary, PATH USER POSITION (R4-3 or R4-1) signs may be installed to remind users of this required behavior (see Figure 14-34) **(33)**.

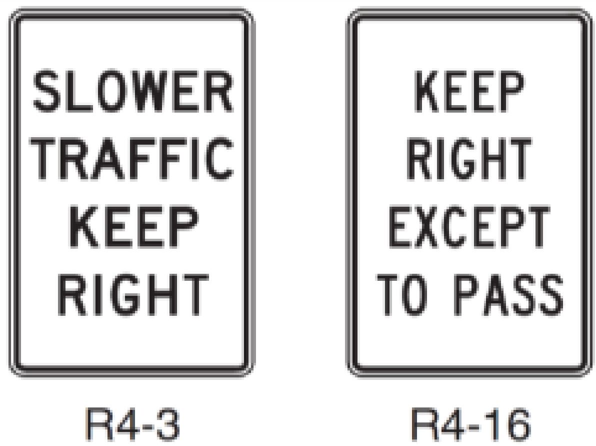


Figure 14-34 Path User Position Signs

In cases where there are high path volumes it may be appropriate to separate directions on the path with a yellow centerline stripe. On areas with adequate sight distance a broken line (3-foot segment with a 9-foot gap) may be provided.

On the approach to conflict points, substandard curves, locations where sight distances cannot be maintained, or other potential hazards, a single solid yellow centerline stripe and an appropriate sign should be installed. The solid stripe should extend a distance at least equal to the stopping sight distance in advance of the conflict point or hazard.

Where users are split onto separate paths, mode specific guide signs should be used to denote the preferred path for each user type (see Figure 14-36). SELECTIVE EXCLUSION signs **(33)** can be used to indicate where various users are not permitted (see Figure 14-35).

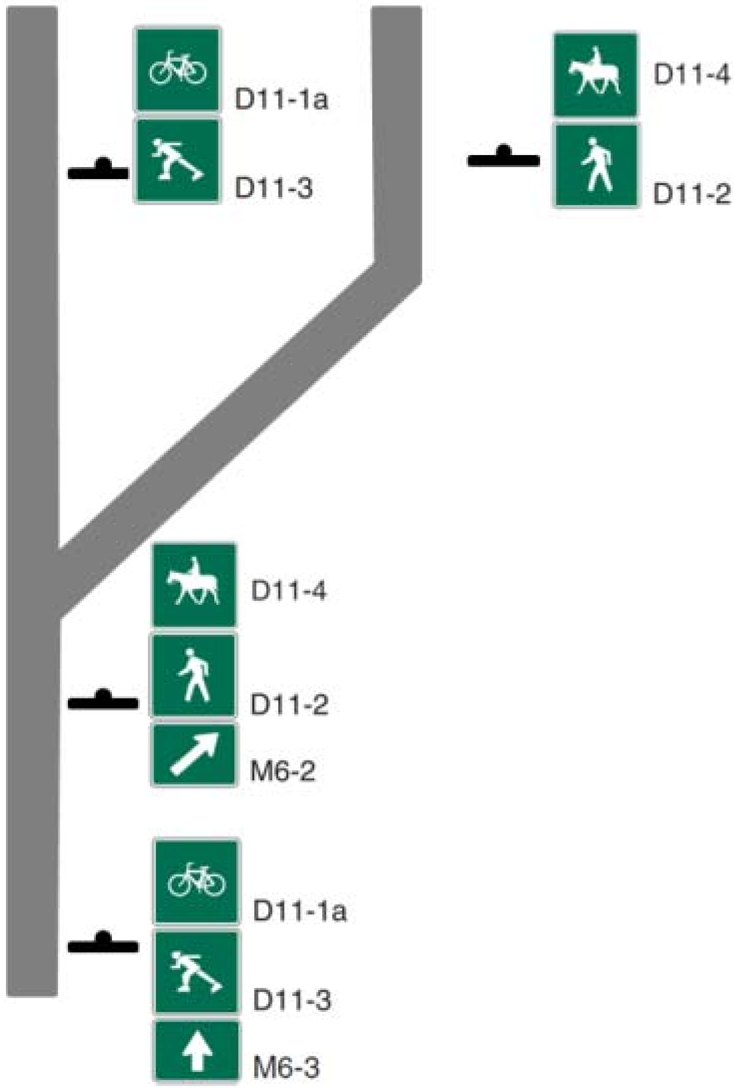


Figure 14-35 SELECTIVE EXCLUSION Signs

Figure 14-36 Mode Specific Guide Signs

14.2.5 Cross Slope

The cross slope of a shared use path must be designed so that rain and snow melt will drain from the pavement surface. Consequently, a minimum cross slope of 1 percent should be maintained on shared use paths. Shared use paths typically are not crowned; a uniform cross slope is maintained across the path.

Because shared use paths are intended to be used by pedestrians and persons with disabilities, they must comply with the cross slope requirements of the ADA. Therefore, the maximum cross slope for a shared use path is 2 percent.

14.2.6 Clearances

Just as minimum clear recovery areas and clear zones to obstructions are provided for roadways, horizontal clearance is required to signs, poles, drop-offs and other path-side obstructions and hazards.

Where practical, a graded shoulder free of obstructions at least 3 feet wide with a maximum cross slope of 6:1 should be maintained on each side of the shared use path pavement. Under constrained conditions, minimum clear space of 2 feet should be provided to vertical obstructions. If a smooth protective railing is provided, this clearance may be reduced to 1 foot. Where minimum clearance cannot be provided to obstructions, path users should be warned of the upcoming obstruction. Warnings for lateral obstructions can include warning signs, edge line striping, reflectorization, or a combination thereof. When a barrier, railing, or fence is a vertical obstruction, the barrier should be flared so the approach end is at least 3 feet from the edge of the path.

Embankments and sheer drop-offs are particularly hazardous to shared use path users. If possible a 5-foot separation should be provided to embankments with slopes greater than 4:1 and drop-offs. Where this separation cannot be maintained, a suitable barrier such as a railing or fence should be provided at the top to the slope. Specifically, barriers should be placed to separate shared use paths from embankments and drop-offs under the following conditions (see Figure 14-37):

- Slopes 3:1 or steeper, with a drop of 6 feet or greater
- Slopes 2:1 or steeper, with a drop of 4 feet or greater
- Slopes 1:1 or steeper, with a drop of 1 foot or greater
- Slopes 3:1 or steeper, adjacent to a parallel water hazard, roadway, or other obvious hazard

When used, barriers next to a shared use path shall be a minimum of 42 inches high.

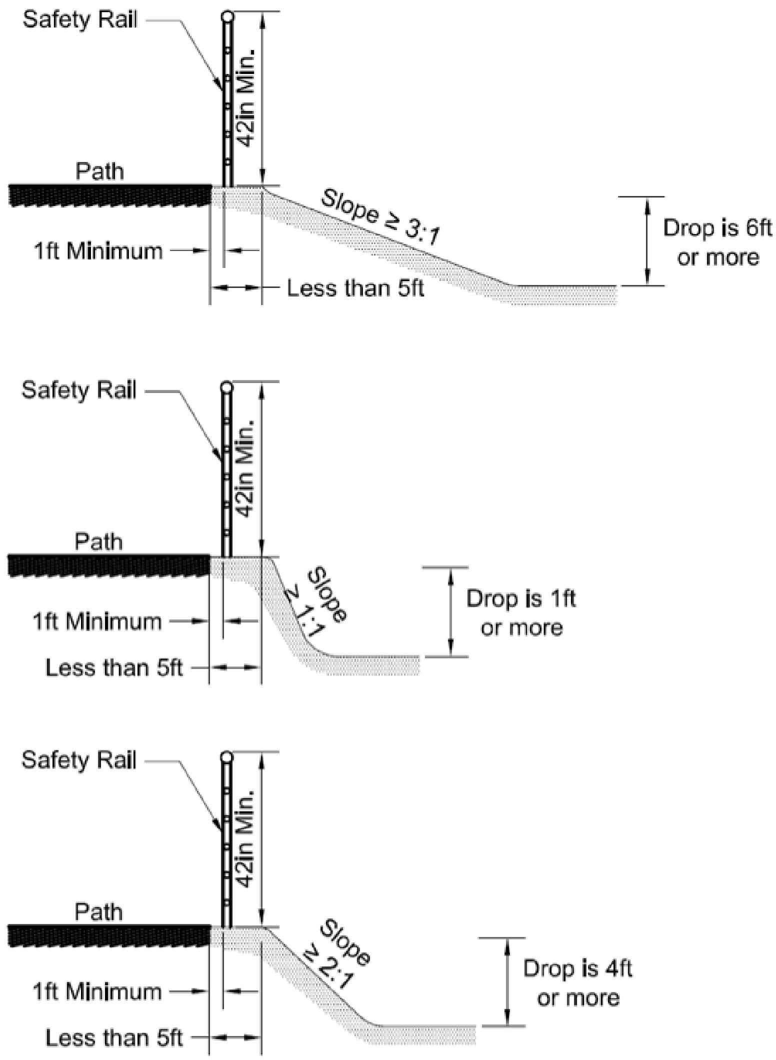


Figure 14-37 Conditions where barriers to embankments are recommended

Openings between horizontal or vertical members on railings should be small enough that a 4-inch sphere cannot pass through them in the lower 27 inches. For the portion of railing that is higher than 27 inches, openings may be spaced such that an 8-inch sphere cannot pass through them. This specification is to prevent children from falling through the openings.

Some Colorado jurisdictions require a rub rail at a height where a bicyclist's handlebar may come into contact with a railing or barrier. A rub rail is a smooth surface 36 inches to 44 inches installed to reduce the likelihood bicyclists' handlebars will be caught by the railing. Local requirements should be consulted.

The minimum vertical clearance to obstructions is 100 inches, the operating height for a bicyclist.

14.2.7 Horizontal Alignment of Shared Use Paths

The discussion of horizontal alignment provided in Chapter 3 is also applicable to shared use paths. Typically, simple horizontal curves should be used on shared use paths.

Because a shared use path is also a pedestrian facility, paths must be designed to be compliant with the applicable sections of the ADA. Consequently, the maximum superelevation allowed on a shared use path is 2 percent. If separate pathways for pedestrians and bicyclists are provided, the superelevation allowed for the bicycle path may be increased up to 8 percent.

The minimum radius recommended for shared use paths is provided in Table 14-7

If the minimum curve radius cannot be met, a centerline stripe and TURN or CURVE WARNING sign (W1 series) shall be installed.

The AASHTO Bicycle Guide provides an alternative method for calculating minimum radii which in some cases yields a smaller required radius. It is based upon the lean angle of a bicycle.

e	**R** (feet) **for Design Speed** (mph)								
(%)	8	10	12	14	16	18	20	25	30
-2.0	14	22	33	47	64	85	109	192	316
-1.5	14	22	33	46	63	83	107	188	308
0.0	13	21	31	44	60	79	101	176	286
1.5	12	20	30	42	57	74	96	165	267
2.0	12	20	29	41	56	73	94	162	261
2.2	12	20	29	41	55	73	93	161	259
2.4	12	19	29	41	55	72	93	160	256
2.6	12	19	29	40	55	72	92	158	254
2.8	12	19	29	40	54	71	91	157	252
3.0	12	19	28	40	54	71	91	156	250
3.2	12	19	28	40	54	70	90	155	248
3.4	12	19	28	40	53	70	89	154	246
3.6	12	19	28	39	53	69	89	153	244
3.8	12	19	28	39	53	69	88	151	242
4.0	12	19	28	39	52	69	88	150	240
4.2	11	19	27	39	52	68	87	149	238
4.4	11	18	27	38	52	68	87	148	236
4.6	11	18	27	38	51	67	86	147	234
4.8	11	18	27	38	51	67	85	146	233
5.0	11	18	27	38	51	66	85	145	231
5.2	11	18	27	37	51	66	84	144	229
5.4	11	18	27	37	50	66	84	143	227
5.6	11	18	26	37	50	65	83	142	226
5.8	11	18	26	37	50	65	83	141	224
6.0	11	18	26	37	49	64	82	140	222
f = Friction Factor	0.33	0.32	0.31	0.30	0.29	0.28	0.26	0.24	0.21

Table 14-7 Minimum Radii and Superelevation for Bicycle Only Paths

14.2.8 Vertical Alignment of Shared Use Paths

Where technically feasible, the maximum continuous grade on a shared use path should be limited to 5 percent. Where right-of-way and geometric constraints make the provision of a continuous grade less than 5 percent impractical, grades should be minimized.

Where potential grades exceed 5 percent, intermittent level resting intervals should be considered. Where provided, resting intervals shall be full width of the shared use path and 60 inches long. Alternatively, a 36-inch wide resting interval may be located adjacent to the shared use path. Recommended maximum distance between resting areas is 200 feet.

Shared use paths located along roadways may follow the grade of the road. Where grades exceed 5 percent, resting intervals should be provided.

Where sustained grades exceeding 4 percent in excess of 300 feet in length are required, an increased design speed should be used. Additionally, consider providing the following mitigating measures:

- HILL WARNING signs (W7-5) (Figure 14-38);
- Wider clear recovery areas adjacent to the shared use path; and
- An additional 6 feet of width to allow some users to dismount and walk their bicycles.

Figure 14-38 Bicycle HILL WARNING Sign

Alternatively, consider installing a series of switchbacks to reduce the longitudinal grade.

Except for ramps on structures, transitions between grades with more than 2 percent algebraic difference should be made with vertical curves. The minimum length for a vertical curve on a shared use path is 3 feet.

On unpaved shared use paths, grades greater than 3 percent are not recommended. Grades exceeding 3 percent can create maintenance (erosion) problems and cause bicycle handling problems for some cyclists.

In flat terrain, the grade of the shared use path may be controlled by drainage considerations.

14.2.9 Intersections with Shared Use Paths

The background information provided in Chapter 9 of this *Roadway Design Guide* is applicable to intersections of shared use paths with roadways or other shared use paths.

The fundamental design of intersections requires that users be able to

- Perceive the intersection and the potential conflicts
- Understand their obligations to yield
- Fulfill the obligation to yield

The design criteria in this section and its subsections are intended to support these three fundamental concepts.

When designing shared use path intersections, the sight distance criteria provided in Section 14.2.3.4 and Chapter 9 of this *Roadway Design Guide* are applicable. Only the design speeds of the intersecting approach legs - using the bicycle as a design vehicle for pathway approaches - are adjusted when applying these criteria to shared use paths.

At shared use path intersections with roadways or with other shared use paths, one facility should be given priority over the other. Four-way stop control should not be used at intersections of shared use paths.

According to the MUTCD **(36)**,

> *When placement of STOP or YIELD signs is considered, priority at a shared use path/roadway intersection should be assigned with consideration of the following:*
>
> > *A. Relative speeds of shared use path and roadway users;*
> >
> > *B. Relative volumes of shared use path and roadway traffic; and*
> >
> > *C. Relative importance of shared use path and roadway.*
>
> *Speed should not be the sole factor used to determine priority, as it is sometimes appropriate to give priority to a high-volume shared-use path crossing a low-volume street, or to a regional shared-use path crossing a minor collector street.*
>
> *When priority is assigned, the least restrictive control that is appropriate should be placed on the lower priority approaches. STOP signs should not be used where YIELD signs would be acceptable.*

The primary consideration in the assignment of traffic control type (STOP as opposed to YIELD signs) at intersections is the availability of adequate sight distance for approaching users. If sight triangles cannot be maintained to provide for yield control, STOP signs must be used. A detailed discussion of sight triangles is provided in Section 14.2.9.1.

Where a shared use path crosses a roadway, detectable warnings shall be installed. Where two shared use paths intersect, the approach that is required to yield the right of way should have detectable warnings installed.

Roundabouts can be used at the intersection of two shared use paths. A width of 8 feet should be maintained around the circulating pathway. Splitter islands and central islands on roundabouts for shared use paths should be curbed.

Traffic control for shared use path approaches to intersections is provided in Section 14.2.9.2.

Intersections of shared use paths with roadways should be located outside of the functional area of the intersection of two roadways (Figure 14-39). If a shared use path crosses a roadway within the functional area of an intersection, the path should either be diverted to outside the functional area of the intersection or moved to the intersection and treated as a sidepath crossing (see Section 14.2.13.1).

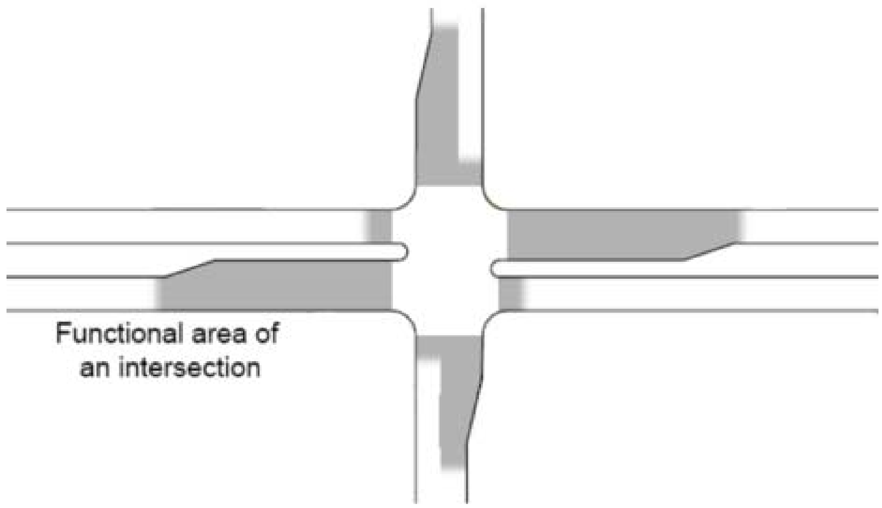


Figure 14-39 Functional Area of an Intersection

Traffic signals can be warranted where shared use paths cross roadways, based on any of the nine warrants described in the MUTCD **(36)**. For the School Crossing and Pedestrian Volume warrants all path users may be counted as pedestrians. For the Eight-Hour Vehicular Volume, Four-Hour Vehicular Volume, and Peak Hour warrants only bicycles are counted as vehicles on the path approaches.

Where signals are installed for shared use paths, signal timing shall accommodate the needs of bicyclists and pedestrians.

14.2.9.1 Required Sight Triangles at Shared Use Path Intersections

The decision to use a STOP sign as opposed to a YIELD sign will be primarily determined by the available sight distance required for bicyclists' at the intersection.

The procedures to determine sight distances at intersections presented in Chapter 9 of the *PGDHS*) **(1)** apply to bicycle facilities as well as to roadways. In this section the requirements for each of the following cases is discussed for both stop and yield control:

Case B3 – Stop Controlled crossing maneuver from the minor road

Case C1 – Yield Controlled crossing maneuver from the minor road

For Case B3 where the path is under stop control, the required sight distance at the intersection is a function of the time it takes the slowest design user to cross the street or cross to a refuge island in the middle of a divided roadway. In most cases the slowest design user is the pedestrian. However, since shared use path crossings of roadways are nearly always marked with crosswalks, the sight distance must allow for a motorist to observe and yield to a pedestrian approaching and crossing at the shared use path-roadway intersection. To calculate the required sight triangle, it should be assumed the pedestrian is standing behind the shared use path yield or stop line.

For Case B3 where the road is under stop control, the sight distance should be calculated as provided in the *PGDSH* **(1)** using the shared use path design speed as the speed on the major road. By applying equation 9-1 from the *PGDSH*

$$ISD = 1.47 V_{path} t_g$$

Where

ISD = intersection sight distance (ft)

V_{path} = design speed of path (mph)

t_g = time gap for minor road vehicle to enter and cross path (sec)

The *PGDSH* provides a time gap, t_g, of 6.5 seconds for passenger cars, 8.5 seconds for single unit trucks, and 10.5 seconds for a combination truck to cross a two-lane roadway based upon observational studies. Consequently, they are conservative for crossing of most shared use paths. However, on multilane roadways where advance STOP or YIELD lines are used, additional time should be allowed: 1.3 seconds additional for a 30-foot advance line and 1.8 seconds for a 50-foot advance line for passenger cars (2.1 seconds and 2.9 seconds for trucks respectively). Additionally, where approach grades exceed 3 percent, add 0.1 second for each percent grade.

The clear sight triangle is that space which should be kept free of obstructions that might block an approaching driver's view of any potentially conflicting path users. Figure 14-40 illustrates the needed dimensions for calculating the sight triangle for case B3 where motorists are required to stop. Table 14-8 provides the values for those dimensions.

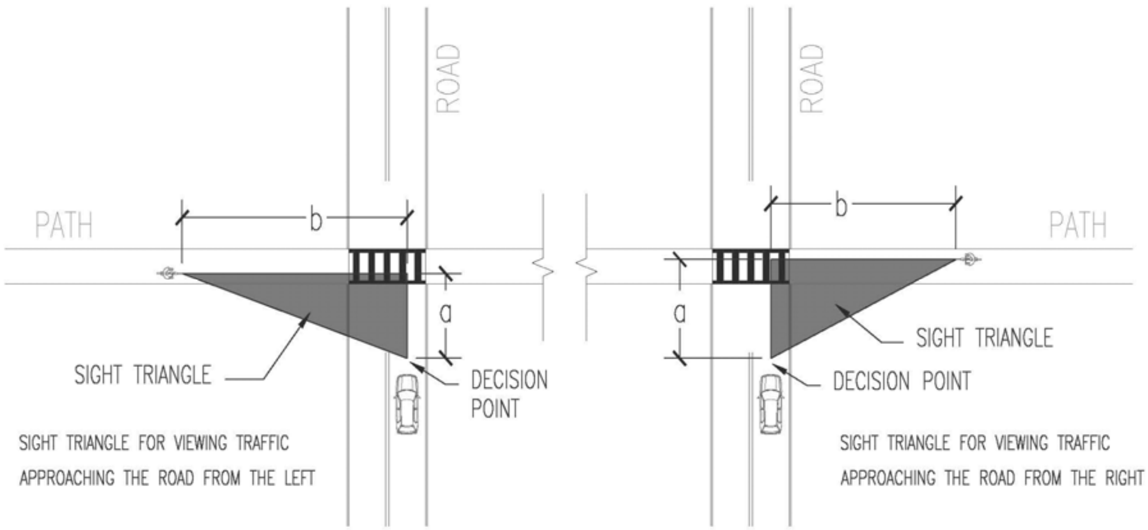


Figure 14-40 Illustration of Intersection Sight Triangle Dimensions

Case B3, Motorist Required to Stop

a = assumed distance to driver's eye

b = intersection sight distance

Design Speed of Path (mph)	Intersection Sight Distance for Passenger Cars (distance *b*) Distance to Stop Bar		
	4 feet	30 feet	50 feet
8	80	95	100
10	100	115	125
12	115	140	150
14	135	165	175
16	155	185	200
18	175	210	220
20	195	230	245
25	240	290	310
30	290	345	370
Asumed distance to driver's eye (distance a)	14.5 feet	40.5 feet	50.5 feet

Table 14-8 Intersection Sight Distance

For Case C3 where the path is under yield control, sight triangles are calculated assuming that the yielding approaches will decelerate to 60 percent of the design speed on the approach to the intersection and that the approaches with priority will not decelerate. Sight distances are calculated based upon the time taken for the vehicle on the minor road to cross the intersection. The travel time to reach and clear the major road from the decision point on the minor approach is calculated using the following equations:

$$t_g = t_a + \frac{w + L_a}{0.88V_{minor}}$$

where

$$t_a = \frac{1.47(V_{minor} - V_r)}{a_m}$$

and

t_g = time gap for minor road vehicle to reach and clear the major road (sec)

t_a = travel time for minor road vehicle to reach the major road while decelerating (sec)

w = width of intersection to be crossed (ft)

L_a = length of design vehicle (ft)

V_{minor} = design speed of minor facility (mph)

V_r = reduced speed of minor approach (60 percent design speed)(mph)

a_m = acceleration rate assumed for minor approach (assume 5 ft/sec/sec)

The length of the sight triangle along the major approach is calculated using the equation

$$b = 1.47V_{major}t_g$$

where

b = sight distance required along major approach (ft)

V_{major} = design speed of major facility (mph)

The sight distance required along the minor approach, a, can be obtained from Table 14-9.

Figure 14-41 illustrates the dimensions for yield control intersections. Users are not shown on the graphic because either approach (major or minor) could be the shared use path.

Design Speed of	Minor Leg
12	62
14	71
16	80
18	90
20	100
25	130
30	160
35	195
40	235
45	275
50	320
55	370

Table 14-9 Required Sight Distance for Minor Leg of Yield Control

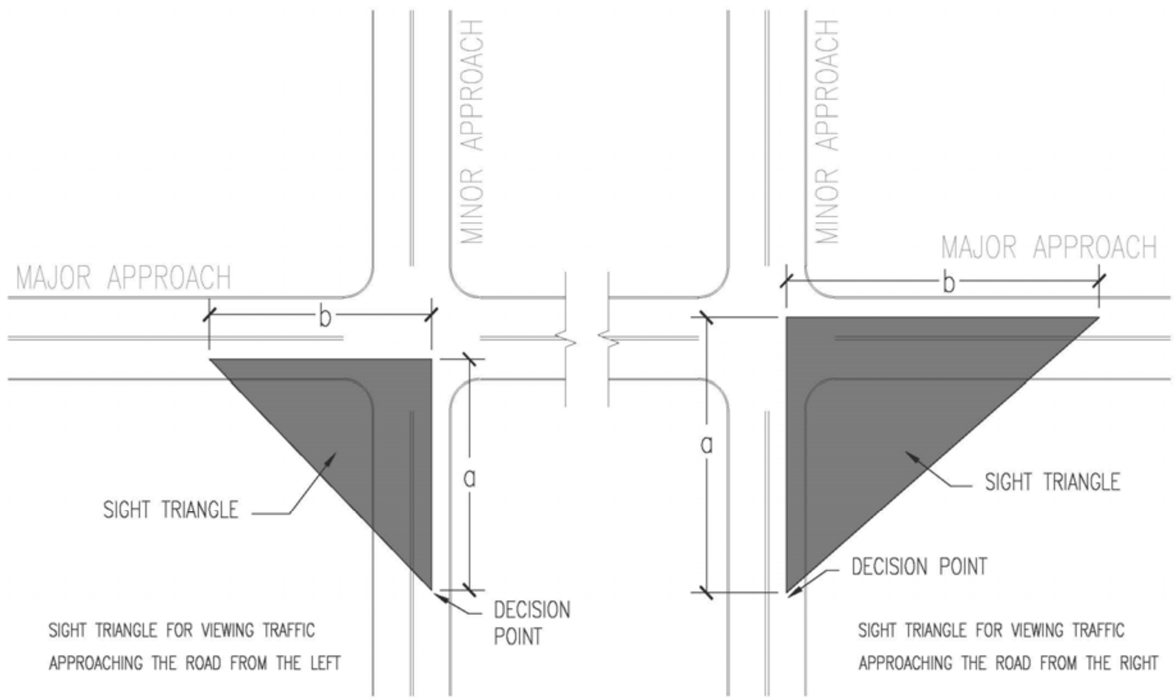


Figure 14-41 Illustration of Intersection Sight Triangle Dimensions. Case C3, Yield Condition

Where a shared use path approaches a walkway and is required to stop, the legs of the sight triangle should extend 25 feet back from the edge of the sidewalk along the shared use path, and 15 feet back from the edge of the shared use path along the sidewalk (Figure 14-42).

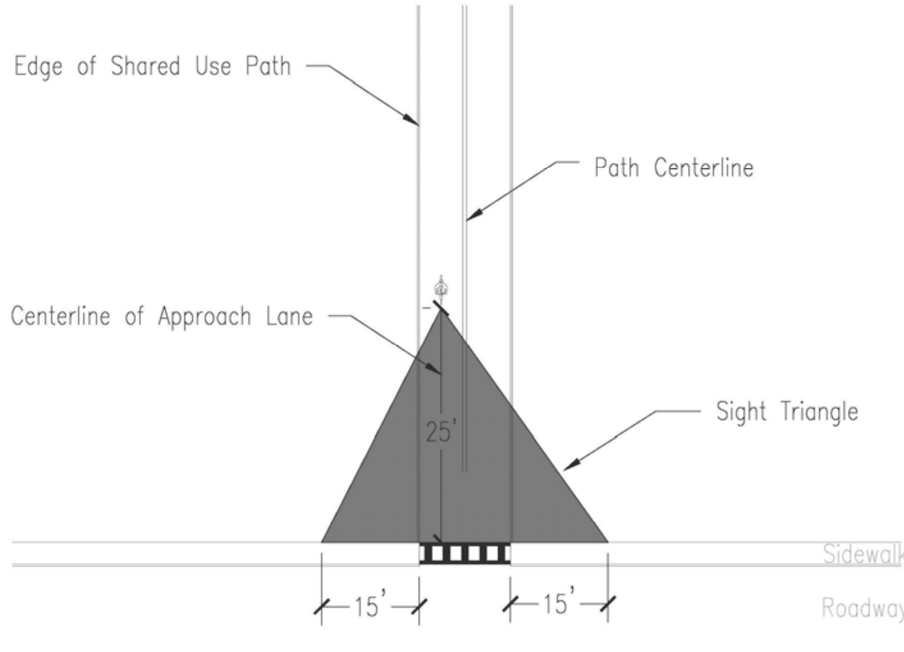


Figure 14-42 Illustration of Intersection Sight Triangle Dimensions. Path Approaching Sidewalk

14.2.9.2 Traffic Control at Intersections with Shared Use Paths

The traffic control provided on shared use paths at intersections with other paths or with roadways is similar to that provided at the intersection of two roadways.

STREET NAME signs (D1-3) should be included for shared use path users.

On the approach to any intersection, a solid yellow centerline should be striped on the approach to the intersection for a distance equal to the stopping sight distance of the shared use path.

An INTERSECTION WARNING (W2 series) or ADVANCE TRAFFIC CONTROL (W3 series) sign may be used on a roadway, street, or shared-use path in advance of an intersection to indicate the presence of an intersection and the possibility of turning or entering traffic. However, these signs are not required unless the engineering judgment determines that the visibility of the intersection is limited on the shared-use path approach to the intersection. When deciding whether to install advance signs, the designer should ensure that intersections and intersection traffic control are visible from at the least stopping sight distance in advance of the intersection. Figure 14-43 shows W2 and W3 series signs.

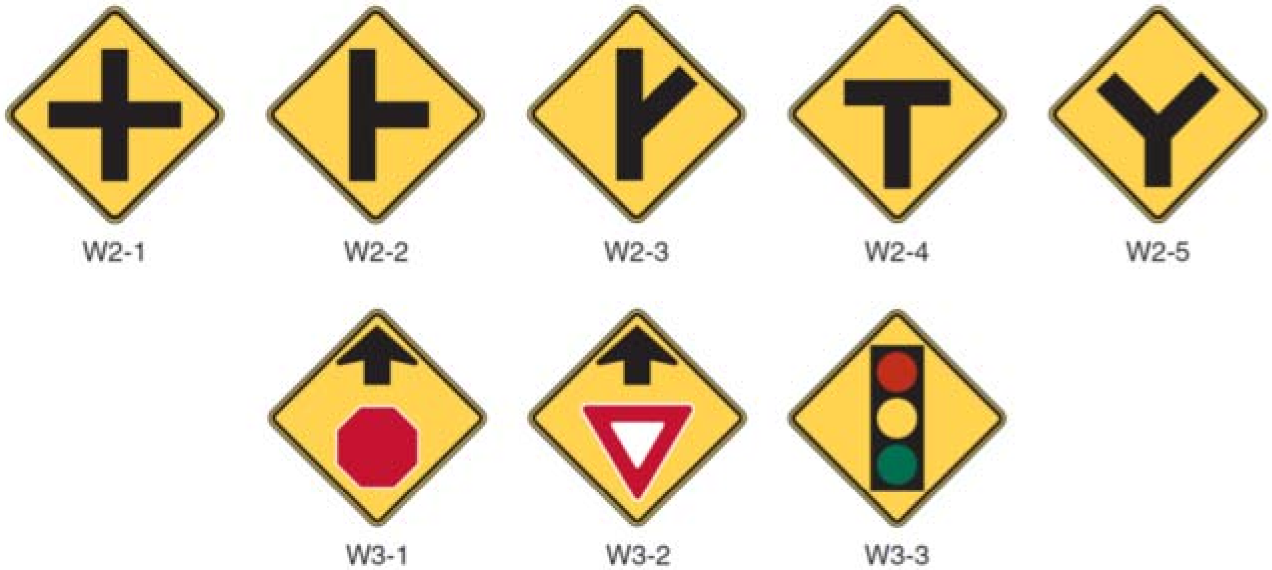


Figure 14-43 INTERSECTION WARNING (W2 Series) and ADVANCE WARNING SIGNS (W3 Series) Signs

Where the shared use path user is to yield or stop (with either a STOP sign or a signal) at the intersection, YIELD signs and YIELD lines or STOP signs and STOP lines shall be installed on the path approach to the intersection. YIELD or STOP lines shall be placed 4 feet in advance of the intersecting travel way or sidewalk.

For signal control intersections, detector loops in the pavement and push buttons for pedestrians should be installed on the path approaches.

On the motor vehicle approach, signing and striping will vary depending on which facility is given priority at the intersection. If the path is given priority at the intersection, then the roadway approaches should be signed and marked as they would be on the approach to any intersection with with similar control (YIELD, STOP, or signal control). If the roadway is given priority at the intersection, traffic control appropriate for a midblock crossing must be installed (see Sections 14.3.8 and 14.3.9). At trail crossings, the TRAIL CROSSING (W11-15 and W11-15p) sign assembly (Figure 14-44) should be used instead of the PEDESTRIAN CROSSING sign (W11-2).

At any activated crossing (e.g., a hybrid beacon), if the bicyclists is required to cross the roadway in stages, additional activation mechanisms (i.e., loops, video detection, push buttons) must be placed in the median. Signing should be provided to make bicyclists aware of any requirement on their part to activate multiple crossings.

14.2.9.3 Reducing Speeds on the Approach to Intersections

Figure 14-44 TRAIL CROSSING Assembly

As stated in Section 14.2.8, users of intersections must be able to perceive a conflict, understand their obligation, and be able to fulfill their obligation to yield or stop. Slowing drivers and path users down on the approach to intersections can provide more time for users to perceive and understand their obligations.

Horizontal deflection, either through a series of low design speed curves or a chicane, on the approach to an intersection is an effective technique to reduce bicycle speeds. Examples of these geometric design techniques are provided in Figure 14-46 and Figure 14-45.

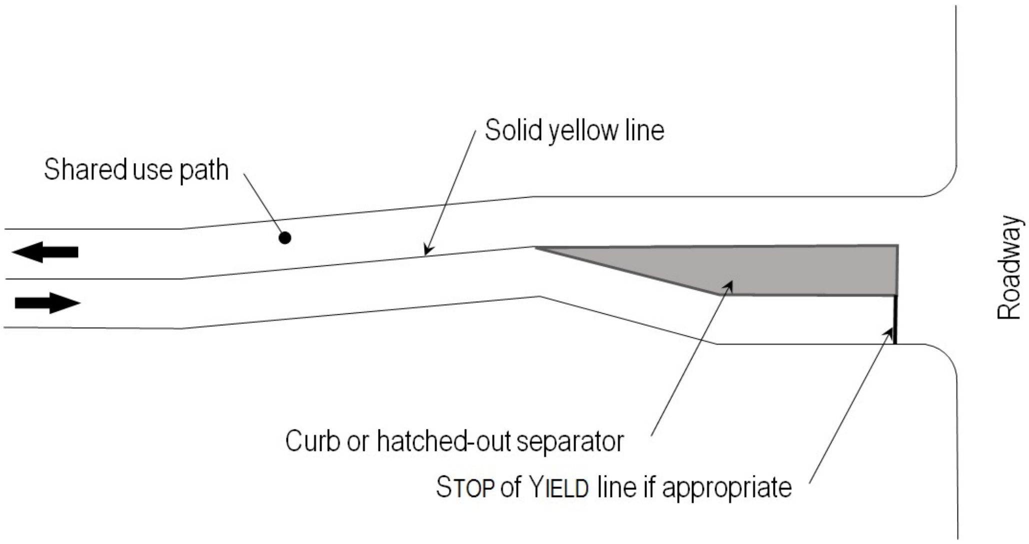


Figure 14-45 Chicane on Approach to Intersection

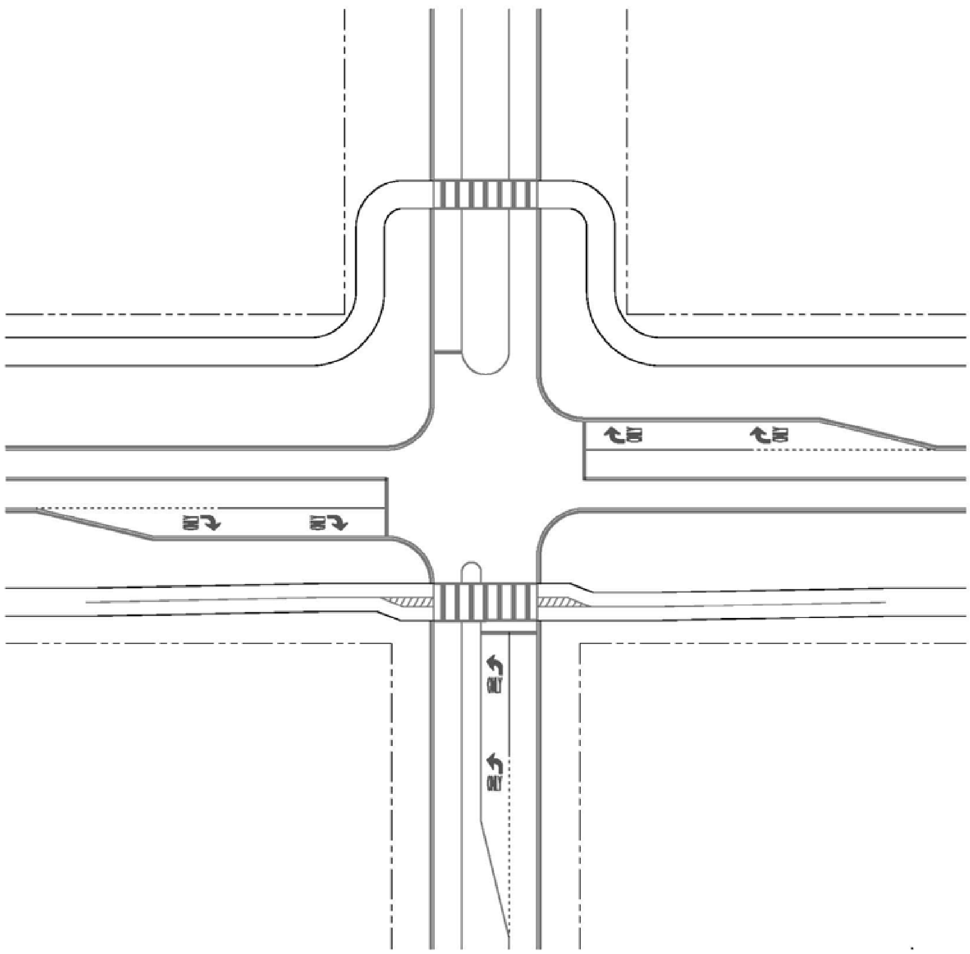


Figure 14-46 Geometric Design to Slow Bicyclists on Intersection Approaches

14.2.9.4 Curb Ramps

Anytime a shared use path crosses a roadway it is a pedestrian crossing location. ADA compliant curb ramps (if curbs are present) must be installed. The width of the ramp, not inclusive of the flares or curb returns, must be the full width of the approach path. Refer to Section 14.3.1.4 of this chapter.

Detectable warnings must be placed at the base of the curb ramps across the entire width of the ramps or across the entire width of the path on the approach to crossings where no curbs are present.

14.2.9.5 Prevention of Motor Vehicle Encroachment onto Shared Use Paths

On some shared use paths, encroachment by motor vehicles may be a concern. If the primary cause of encroachment is a lack of understanding on the part of motorists of the non-motorized nature of the facility, consider the installation of NO MOTOR VEHICLES (R5-3) (see Figure 14-47) signs at path access points.

Figure 14-47 NO MOTOR VEHICLES Sign (R5-3)

Physical barriers to motor vehicles are often ineffective in prohibiting access to motor vehicles. Motorists, and more frequently all-terrain vehicles, often go around or damage objects intended to limit motor vehicle access. Barriers can, however, present obstructions to shared use path users. Consequently, their use should be limited.

One method of discouraging access to motorists is the use of a low, central, dividing island on the path approach to intersections. Combined with tight curb radii, this method can be quite effective. The island should be designed so that emergency and maintenance vehicles can access the path by straddling the island. The width of the path on either side of the island should be at least 6 feet wide; in constrained conditions the path may be narrowed to 5 feet wide on either side of the dividing island. Where divisional islands are provided, solid yellow lines are to be provided in advance of and on either side of the island.

Tight curb radii, such as 2 feet, at path-roadway intersections can reinforce the non-motorized nature of shared use paths. See Figure 14-48.

If bollards are used to restrict motor vehicle access at intersections of roadways with paths, a 6 foot clear space is to be provided between bollards. If more than one bollard is used, then an odd number of bollards shall be used so that one bollard is in the center of the path. Obstruction striping shall be installed around bollards. Around the *central* bollard the obstruction striping shall be yellow to denote opposing directions of travel on either side of the bollard. Additional bollards shall have white obstruction markings. See Figure 14-49. Solid lines on the approach to the bollards should extend a distance equal to the stopping sight distance in advance of the bollards.

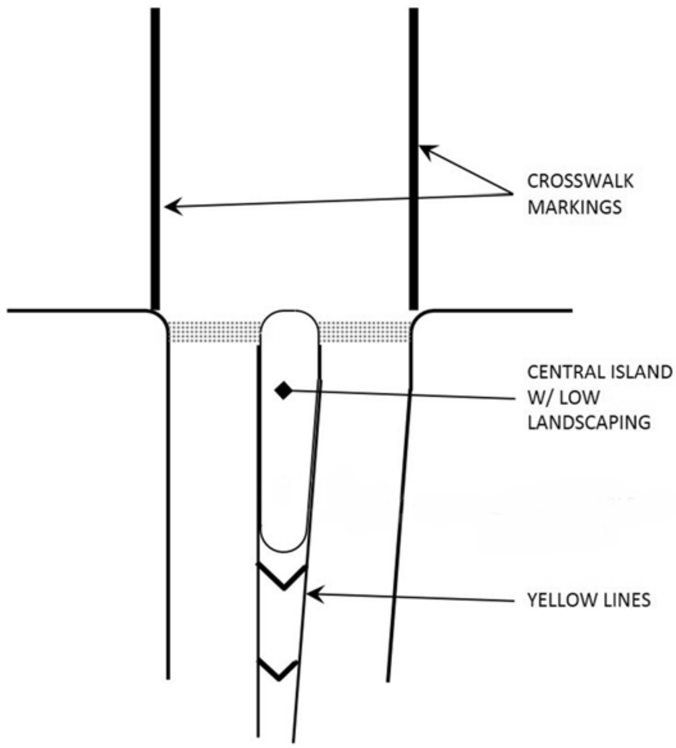


Figure 14-48 Example of Schematic Path Entry

Directional arrows may be placed on the approach to the paths between bollards to prevent confusion of path users. Where used, bollards shall be marked with retroreflective material on both sides or the appropriate object marker as shown in the *MUTCD* (37). In addition, bollards should be:

- Visible from a distance equal to or greater than the stopping sight distance
- At least 40 inches high
- Have a minimum diameter of 4 inches
- Be set back 30 feet from the through lanes on the adjacent roadway.

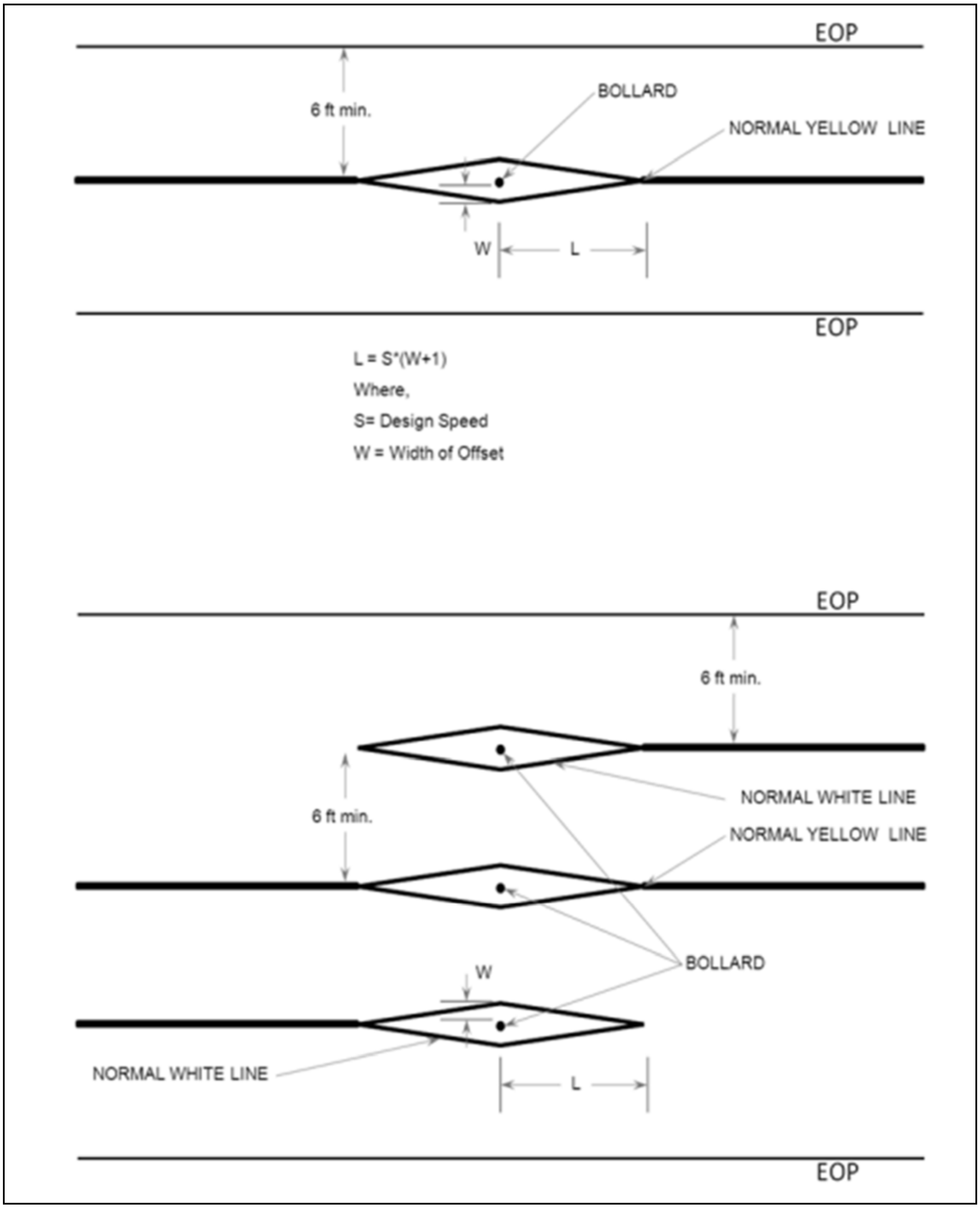


Figure 14-49 Obstruction Striping around Bollards on Shared Use Paths

If used, bollards shall be placed where motorized vehicles cannot easily bypass them.

Bollards should be installed in such a manner as to be removable by emergency or maintenance personnel. Any hardware used to secure the bollard should be flush with the surface of the bollard or ground so as not to create an additional obstruction.

14.2.10 Underpass and Overpass Structures

To maintain the continuity of a shared use path some structures may be required. When a designer has to choose between a tunnel and an overpass the characteristics of each crossing should be considered before determining which structure type is most appropriate. Each structure type has benefits and drawbacks which need to be considered for each individual location. Constraints such as right-of-way, topography, and utility conflicts may dictate whether an overpass or underpass is more appropriate.

Overpasses have several benefits. Overpasses generally provide good visibility of surrounding areas which may lead to a greater sense of security, they are well lit during daylight hours, and they more easily accommodate drainage. Conversely, overpasses typically require a greater elevation change and may be more difficult for users to traverse, they are exposed to the elements, and speeds on the downward approaches can be hazardous.

Underpasses often exhibit contrasting characteristics to overpasses. They are protected from the elements and often require less ramping or changes in elevation, typically making them easier to traverse. Underpasses, if not designed properly, can be dark and intimidating and may feel claustrophobic. Underpasses also often present drainage challenges, utility conflicts, and construction phasing issues. Underpasses will often require lighting and additional maintenance such as regular sweeping to remove sedimentation.

Underpass design and layout should consider user safety. Limited visibility through a closed structure may have a negative impact on user's perception of personal safety. When an underpass is long, wider openings, additional width, or flared ends may be appropriate to improve natural lighting and visibility. Approaches and grades should be evaluated to provide the maximum possible field of vision towards the underpass.

14.2.10.1 Width and Clearance for Structures Serving Shared Use Paths

All bridges and tunnels serving shared used paths should carry the width of the approach path and the minimum clear space of 2 feet on each side of the path across the structure. Carrying the clear space across the structure provides maneuvering space to avoid pedestrians or stopped bicyclists, as well as necessary horizontal shy distance from railing, walls, or barriers.

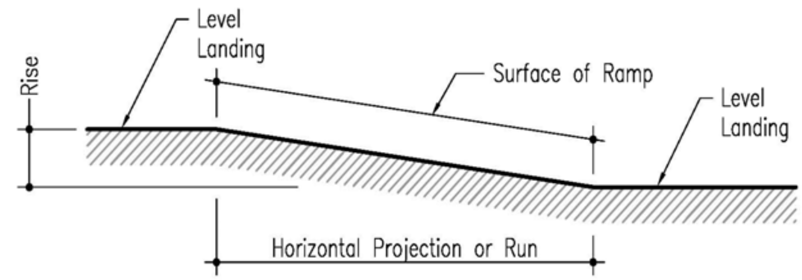
If the full path width and clear space cannot be carried across a structure, railings with proper end flares should be provided to reduce the path width on approaches (see Section 14.2.6).

Access by emergency or maintenance vehicles should be considered when establishing the clearances of structures serving shared use paths. Motor vehicles authorized to use the path may dictate the vertical and horizontal clearances.

A vertical clearance of 10 feet is desirable for enclosed structures and tunnels. If access for motor vehicles is not required then the minimum vertical clearance provided shall be 8 feet under constrained conditions. Designers may want to consider providing 8.3 feet (100 inches), which is the operating height of a bicyclist, when on a shared use facility **(2)**.

14.2.10.2 Grades on Structures Serving Shared Use Paths

All structures serving shared use paths must be ADA compliant. Cross slopes shall not exceed 2 percent. If approach grades exceed 5 percent they shall be designed as ramps. Resting intervals measuring 60 inches in the direction of travel along the path and full width of the structure shall be provided a maximum of every 30 inches of rise. See Figure 14-50.



Slope	% Grade	Maximum Rise In.	Maximum Horizontal Projection Ft.
1:12 to > 1:16	8.33% to > 6.25%	30	30
1:16 to > 1:20	6.25% to > 5.00%	30	40

Components of a Single Ramp Run and Sample Ramp Dimensions

Figure 14-50 Maximum Spacing of Resting Intervals on Shared Use Path Structure Ramps

14.2.10.3 Railings on Structures Serving Shared Use Paths

Railings on shared use path structures shall be designed to comply with Section 14.2.6 of this chapter.

14.2.10.4 Railroad Crossings

Where possible, shared use paths should be aligned to cross railroad tracks at near right angles. Where this cannot be accomplished and the crossing angle is less than 45°, SKEWED CROSSING signs (W10-12) shall be placed on the path approaches to the rail crossing.

A railroad-path crossing, like a railroad-highway crossing, involves either a separation of grades or a crossing at-grade. The horizontal and vertical geometrics of a path approaching an at-grade railroad crossing should be constructed in a manner that does not divert a path user's attention from path surface conditions.

The same types of crossing treatments used for roadway crossings of railroads, ranging from the required CROSSBUCK sign (R15-1) to full signals and gates, can be used on shared use paths.

Where active traffic control devices are not used, a CROSSBUCK ASSEMBLY shall be installed on each approach to a pathway grade crossing. The CROSSBUCK ASSEMBLY may be omitted at station crossings and on the approaches to a pathway grade crossing that are located within 25 feet of the traveled way of a highway-rail or highway-LRT grade crossing. Pathway grade crossing traffic control devices shall be located a minimum of 12 feet from the center of the nearest track.

If used at a pathway grade crossing, an active traffic control system shall include flashing-light beacons for each direction of the pathway. A bell or other audible warning device shall also be provided.

Advance pavement markings and signs shall be used on the approach to railroad crossings. See Figure 14-51. The minimum sizes of pathway grade crossing signs shall be as shown in the shared-use path column in Table 9B-1 of the *MUTCD*.

If used, swing gates shall be designed to open away from the tracks so that pathway users can quickly push the gate open when moving away from the tracks. If used, swing gates shall be designed to automatically return to the closed position after each use.

To meet the requirements of the draft Public Right of Ways Accessibility Guideline (PROWAG), path surfaces shall be flush with the tops of rails ***(48).*** Openings for wheel flanges at path crossings of freight rail track shall be 3 inches maximum. Openings for wheel flanges at path crossings of non-freight rail track shall be 2.5 inches maximum.

Coordinate early and often with the railroads to determine the appropriate design elements.

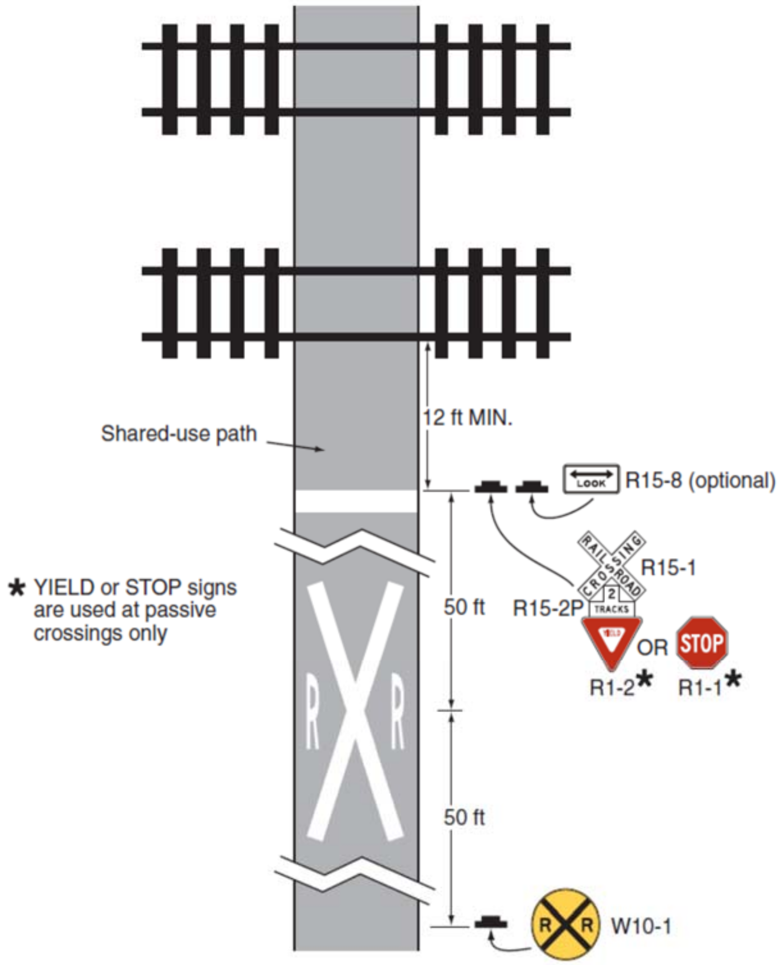


Figure 14-51 Example Signage and Markings at a Shared Use Path Crossing of a Rail Road (49)

14.2.10.5 Utilities

As discussed in Section 14.1.2.10.1, drainage structures and utility lids should not be placed in shared use paths. Where it is unavoidable, drainage grates shall be of a bicycle friendly design and utility covers shall be flush with the surface of the path (see section 14.1.12.2 for examples of bicycle friendly grates).

Utilities that project from the ground, such as backflow preventers or valves, shall be treated as vertical obstructions and addressed as discussed in Section 14.2.6 (Clearances).

14.2.10.6 Traffic Calming on Shared Use Paths

In some situations, such as in areas with frequent crossing conflicts with motor vehicles, it may be desirable to limit the speed traveled by the path user (see section 14.2.2 Design Speed). Signing is not an effective method for reducing speeds for two reasons: (1) because bicyclists, like motorists, ride at a speed they feel comfortable with on a facility, and (2) because most bicyclists do not have speedometers installed on their bicycle. Consequently, the use of design features is recommended to reduce speeds on shared use paths.

Vertical traffic calming treatments (speed humps, tables or pillows) are not recommended on shared use paths as they can adversely impact the handling of wheeled operators.

Horizontal alignment is the recommended method of reducing speeds on shared use paths. A series of low design speed curves or a chicane along a path, much like those described in Section 14.2.9.4 (Reducing Speeds on the Approach to Intersections), can also be used to reduce speeds at non intersection areas. Advance striping and signage should supplement the trail calming features, either appropriate CURVE WARNING signs or a general text sign indicating that the section of trail is a reduced speed zone.

14.2.11 Wayfinding on Shared Use Paths

The bicycle wayfinding signs described in 14.1.2.1 Bike Routes may be used on shared use paths.

Additional wayfinding signing on shared use paths is often appropriate. On independent alignment paths, information such as the distance between trail heads, or to the next water fountain or restroom facilities are important to path users. Much as Motorists Service signs provide expressway users information on what amenities are available at interchanges, signs may be appropriate to inform path users of the proximity of dining establishments, bike shops, or other destinations of particular interest to path users.

14.2.12 Shared Use Paths Adjacent to the Roadway (Sidepaths)

The term *sidepath* refers to a shared use path located immediately adjacent and parallel to a roadway.

Ideally, shared use paths will be constructed in their own rights-of-way. However, in some cases a shared use path may be designed adjacent to a roadway. Such cases might include:

- Where the public desires a low stress facility to ride on adjacent to a busy or high-speed roadway
- As a temporary facility where a roadway cannot be modified to include bike facilities, and
- As a connecting facility along a longer shared use path.

It is likely the last condition will be the one that most designers are requested to address. As discussed in Section 14.2.13 the perception of a sidepath as a low stress facility does not necessarily equate to it being a safer facility. For reasons of safety or convenience, a sidepath may not be used by more traffic savvy bicyclists. A sidepath should not be considered a permanent alternative to an in-street facility; rather it should be considered either temporary, or a supplemental facility to serve a specific class of user.

All design criteria associated with shared use paths apply to sidepaths.

14.2.13 Safety Considerations of Sidepaths

Locating a sidepath immediately adjacent to a roadway can create operation concerns. The AASHTO Bike Guide summarizes many of the problems which may occur in Section 5.2.2. A brief synopsis of the more prevalent concerns are as follows:

- Unless separated, they require one direction of bicycle traffic to ride against motor vehicle traffic, contrary to normal rules of the road.
- When the path ends, bicyclists going against traffic will tend to continue to travel on the wrong side of the street. Likewise, bicyclists approaching a shared use path often travel on the wrong side of the street in getting to the path. Wrong-way travel by bicyclists is a major cause of bicycle/automobile crashes and should be discouraged at every opportunity.
- At intersections, motorists entering or crossing the roadway often will not notice bicyclists approaching from their right, as they are not expecting contra-flow vehicles. Motorists turning to exit the roadway may likewise fail to notice the bicyclist. Even bicyclists coming from the left often go unnoticed, especially when sight distances are limited.
- Signs posted for roadway users are backwards for contra-flow bike traffic; therefore these cyclists are unable to read the information without stopping and turning around.
- When the available right-of-way is too narrow to accommodate all highway and shared use path features, it may be prudent to consider a reduction of the existing or proposed widths of the various highway (and bikeway) cross-sectional elements (i.e., lane and shoulder widths, etc.). However, any reduction to less than AASHTO Green Book 1 (or other applicable) design criteria must be supported by a documented engineering analysis.
- Many bicyclists will use the roadway instead of the shared use path because they have found the roadway to be more convenient, better maintained, or safer. Bicyclists using the roadway may be harassed by some motorists who feel that in all cases bicyclists should be on the adjacent path.
- Although the shared use path should be given the same priority through intersections as the parallel highway, motorists falsely expect bicyclists to stop or yield at all cross-streets and driveways. Efforts to require or encourage bicyclists to yield or stop at each cross-street and driveway are inappropriate and frequently ignored by bicyclists.
- Stopped cross-street motor vehicle traffic or vehicles exiting side streets or driveways may block the path crossing.
- Because of the proximity of motor vehicle traffic to opposing bicycle traffic, barriers are often necessary to keep motor vehicles out of shared use paths and bicyclists out of traffic lanes. These barriers can represent an obstruction to bicyclists and motorists.

Additional potential operational and design problems associated with sidepaths include the following:

- Because utilities are often located in the right-of-way, it can be difficult to meet clearance and radii requirements within the available space.
- In addition to traveling in a direction not expected by motorists exiting driveways or side streets, bicyclists riding on sidepaths are also traveling at speeds significantly greater than those of pedestrians. This makes them less likely to be seen by motorists exiting the side street who may be looking immediately to their right for pedestrians.
- If a sidepath is created in a location where there would otherwise be a sidewalk (i.e., a residential neighborhood or an urban commercial district), higher volumes of pedestrians are likely and thus conflicts with pedestrians are likely to increase. While this concern could be mitigated by widening the path, this may increase bicyclists' speeds in off-peak periods, exacerbating the problem of higher speed cyclists approaching conflict points.
- Most roadways have destinations on both sides of the roadway. Since a sidepath serves only one side of the road, this requires sidepath user to cross the roadway midblock to access their destinations or to cross at intersections and ride on a sidewalk (if available) on the opposite side of the road. The former, while not difficult on low volume, low speed streets can be difficult on higher volume, higher speed roadways where sidepaths are likely to be built. The latter may not be legal in some locations.
- The proximity of sidepaths to the roadway may result in bicyclists riding at night being subject to glare from approaching car headlamps. This can make it difficult for the bicyclist to see hazards on the trail surface.

Operational problems associated with the visibility of the path user by motorists are most likely to be more significant on higher speed, higher volume, multilane roadways where motorists are focused on the motor vehicle traffic in the travel lanes **(20)**.

14.2.13.1 Potential Mitigation Measures to the Operational Challenges of Sidepaths

Despite the safety, operational, and design challenges with sidepath design, there are times when they are unavoidable. They are often the preferred facility of the public. It may not be possible to improve the roadway to provide an adopted target level of bicycle accommodation. Alternatively, they may be the only way to complete a bicycle network or close a gap in an otherwise continuous facility. Consequently, sidepath design must include measures to help minimize the operational challenges described in Section 14.2.13. The following geometric measures are the ones most likely to improve the operations and safety at sidepath conflict points.

- Divert the sidepath away from the parallel roadway at conflict points. Ideally, the path should be moved far enough away to function as a midblock crossing and be provided with the appropriate traffic control. At a minimum enough space should be provided for one vehicle (25 feet) to queue between the roadway intersection and the crossing sidepath.

- Reduce the speeds of users on the sidepath. This can be done through horizontal alignment as described in Section 14.2.9.4.
- Reduce motor vehicle speeds at conflicts points. This can be accomplished by designing for the smallest design vehicle likely to commonly turn at the drive or intersection ***(1)*** and using the minimum radii provided for in Chapter 9 of this *Roadway Design Guide.*
- If feasible, reduce the operating speeds on the adjacent roadway.
- Where possible, eliminate conflicts with motor vehicles. Access management techniques such as reducing the number of driveways or installing raised medians reduces the potential conflict locations.
- Keep sight lines clear to ensure that motorists approaching the conflict can clearly see the path users and so path users can see approaching motorists. This requires limiting parking and landscaping around the conflict points. Proper sight distance should be provided.
- Where side path crossing of a side street cannot be separated from the intersection of the side street and the roadway parallel to the sidepath by at least a car length, the crossing should be designed to be close to the adjacent road.
- At signalized intersections, consider installing blank-out signs, to be activated by path users (i.e., push buttons or loops) to alert motorists of their presence. NO RIGHT ON RED blank-out signs would be appropriate for the near side street approach. YIELD TO PEDS IN CROSSWALK would be appropriate for the adjacent right-turn, through-right, and opposing left-turn movements.

Individually, the above measures may not be sufficient to ensure the safety of sidepath users. It is likely a combination of treatments will be required **(20)**.

An additional measure that should be taken is to provide signage to warn motorists of the adjacent path (see Figure 14-52).

Figure 14-52 Example ADJACENT PATH Sign

Unless they are moved to a midblock location, intersections of sidepaths with side streets and driveways are to be given the same priority as the parallel roadway. Installing STOP or YIELD signs at these locations is not an effective method of slowing or stopping path users at side streets and driveways. If path users perceive the signs as overly restrictive, they will not comply with them. Furthermore, motorists may yield to path users and wave them through in conflict with the sign priority at the intersection. The overuse of these signs may decrease their effectiveness at locations where compliance with STOP or YIELD signs is critical to the path users' safety.

14.2.14 Sidepath Clearance to the Adjacent Roadway

The minimum midblock separation between a roadway and sidepath is 5 feet from the back of curb or from the edge of pavement if no curb is present.

If 5 feet of separation cannot be provided, a suitable barrier should be provided. If placed, the barrier should be consistent with the requirements of Section 14.2.6. The location of the barrier shall not impair sight distance at intersections.

On low speed roadways (45 mph or less), it is not necessary for the barrier to be designed to redirect errant motorists toward the roadway unless other conditions require a crashworthy barrier. If the railing cannot be designed so as to not be a hazard to motorists, it shall be protected by a guardrail or barrier wall.

It is not acceptable to mount a railing on top of a guardrail unless it has been appropriately crash tested.

On higher speed roadways, barriers between roadway and sidepaths shall be crashworthy.

At some locations where the pathway is located more than 5 feet from a roadway, a guardrail may be placed between the roadway and the sidepath to protect motorists from an object in the clear zone. When a guardrail is located within 3 feet of the shared use path the back of the guardrail should be considered a vertical obstruction next to the path.

Snow storage should be considered when designing sidepaths. A separation distance of 8 feet is desirable to accommodate snow storage. Where space is limited, overall road cross-section design must consider the likely amount of removed snow, the space needed to store it, and how snow will be managed. When snow is stored in the separation area between the road and shared-use path, at least three-fourths of the path should remain usable. The placement of barrier between the roadway and the shared use path must consider the needs of snow removal and drainage.

14.2.15 Equestrian Facilities

Equestrian facilities may be included on some shared use path projects. Shared bicycle, pedestrian and equestrian use is relatively common across the country. However, care must be taken when designing these facilities to minimize the potential conflicts between equestrians and other users as horses can startle, compromising safety for their riders and other users. Where possible, separate trails or bridle paths, should be provided for equestrian use.

For a complete discussion of equestrian planning and design, the designer should refer to the USDA document *Equestrian Design Guidebook for Trails, Trailheads, and Campgrounds* ***(38)***.

The criteria contained within this section assumes an equestrian path in the same right-of-way as an adjacent shared use path.

14.2.16 Other Considerations on Bicycle Facilities

14.2.16.1 Shared Use Path Lighting

Where shared use paths are used at night, lighting should be provided at intersections with roadways. If implemented, this lighting should be consistent with requirements for roadway intersections contained in Section 5.0 of the CDOT *Lighting Design Guide*, or as necessary, the AASHTO *Roadway Lighting Design Guide.* The CDOT *Lighting Design Guide* is based upon the AASHTO *Guide* and the IESNA *(Illuminating Engineering Society of North America)* recommended practices.

Even where paths are not open at night it may be advisable to light roadway crossings.

In-street bicycle lanes shall be lit to the same level as the adjacent roadway.

14.2.16.2 Maintenance of Traffic

Neither portable nor permanent sign supports should be located on bicycle facilities or areas designated for bicycle traffic. If the bottom of a secondary sign that is mounted below another sign is mounted lower than 7 feet above a pathway, the secondary sign should not project more than 4 inches into the pathway facility ***(47).***

Bicyclists should not be exposed to unprotected excavations, open utility access, overhanging equipment, or other such conditions. Except for short duration and mobile operations, when a highway shoulder is occupied, a SHOULDER WORK sign (W21-5) should be placed in advance of the activity area. When work is performed on a paved shoulder 8 feet or more in width, channelizing devices should be placed on a taper having a length that conforms to the *MUTCD* requirements of a shoulder taper.

If a designated bicycle route is closed because of the work being done, a signed alternate route should be provided. The *MUTCD* includes approved DETOUR signs for bicycle facilities (Figure 14-53). Bicyclists should not be directed onto a sidewalk or exclusive pedestrian path.

Figure 14-53 Bicycle Facility DETOUR Signs

To maintain the systematic use of the fluorescent yellow-green background for pedestrian, bicycle, and school warning signs in a jurisdiction, the fluorescent yellow-green background for pedestrian, bicycle, and school warning signs may be used in Temporary Traffic Control zones.

14.2.16.3 Integration of Bicycles with Transit

Integration of bicycling with transit can increase the utility and extend the range of both modes. Bicyclists sometimes cite trip length, steep grades, and weather as reasons they do not use bicycling as a mode of transportation. By integrating bicycling and transit services, these barriers (real or perceived) can be overcome.

Bicycle racks on, or bicycle space within, transit vehicles can help integrate bicycling and transit. Providing short and long term bicycle parking **(40)** is a key aspect in making this integration.

Where a change in level occurs at a transit station, some modifications may be considered to make the station accessible to bicyclists. Retrofitting a bicycle channel onto an existing staircase is one technique to improve bicycle access (Figure 14-54).

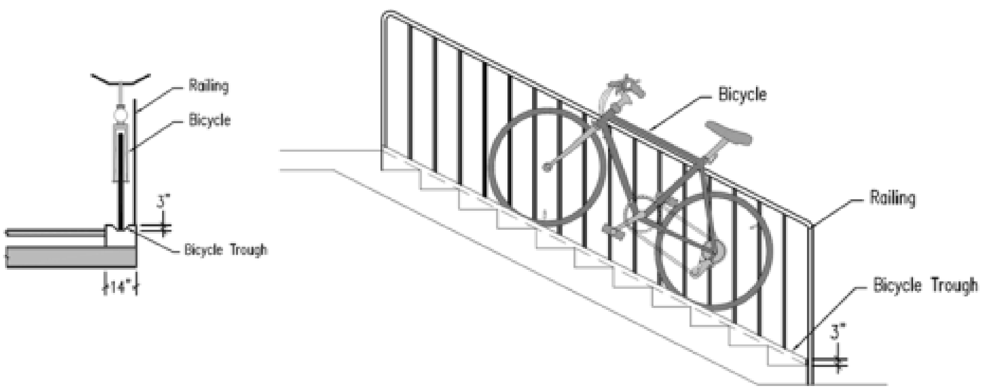


Figure 14-54 Bicycle Channel (41)

Another potential integration of bicycles and transit is use of shared facilities. These are discussed in the following sub-sections.

14.2.16.3.1 Shared Bicycle Facilities with Bus Transit

Shared bicycle facilities with transit can take multiple forms.

Ideally, a bus facility - exclusive busway or bus only lanes - would be constructed with separate bicycle facilities. On an exclusive busway this would entail the provision of a shared use path adjacent the busway (Section 14.2 SHARED USE PATHS). Bicycle lanes can be installed adjacent to, and to the left of, a dedicated bus lane (assuming a right side bus lane).

Alternative facilities can include shared bike-bus lanes. A bike-bus lane can be created by using signing and symbols to allow bicycles to use a designated bus lane (Figure 14-55). A sign similar to the Mandatory Movement Lane Control sign for a bus lane (R3-5gP) could be used. This sign would state that it is a bike lane as well as a bus lane.

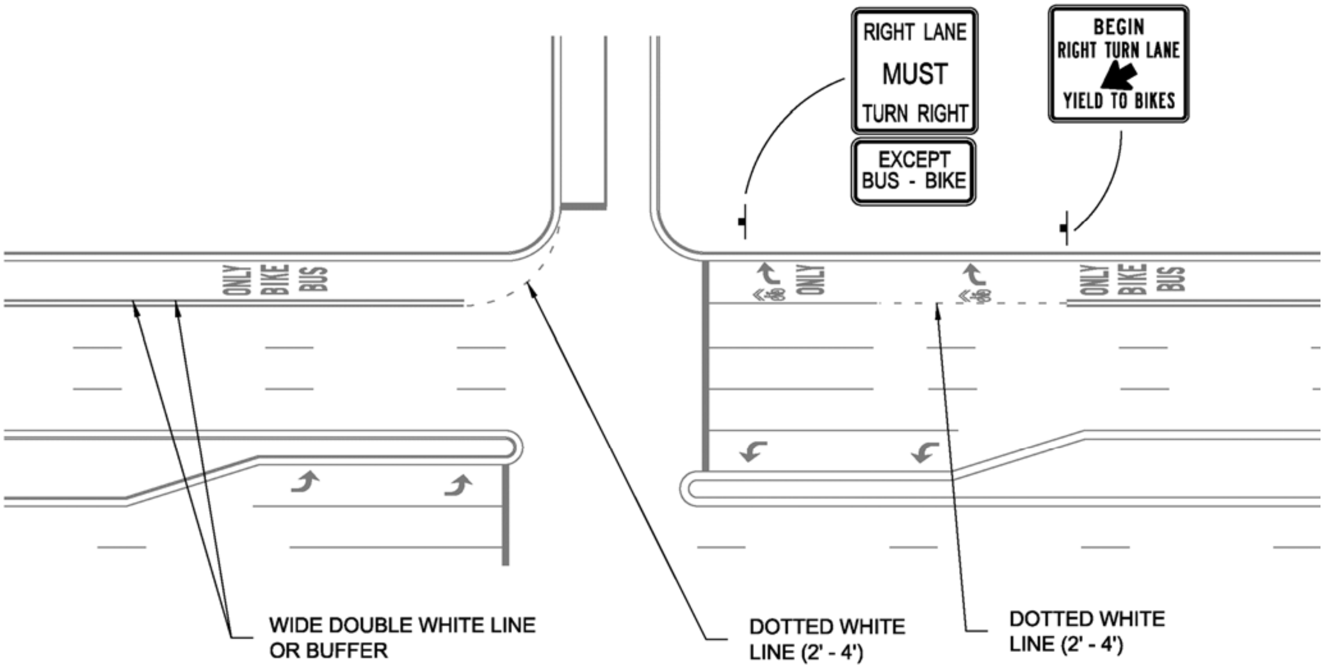


Figure 14-55 Shared Bus Buffered Bike Lane

14.2.16.3.2 Shared Bicycle Facilities with Light Rail

Shared use paths adjacent to rail lines have been implemented around the country.

If shared use paths are constructed adjacent to light rail, special consideration must be given to crossings near rail stops. Treatments to slow bicyclists should be installed in advance of these crossings. Shared use paths adjacent to light rail should be located at least 5-feet clear of the dynamic envelope of the Light Rail Transit vehicle. This will result in the shared use path being at least 11-feet clear of the rail line.

Barriers, as described in Section 14.2.6 (Clearances) should be provided between the light rail facility and the path where practical.

14.2.16.4 Innovative Signing and Markings

Numerous design treatments and traffic control devices are being used or tested to determine their effectiveness in promoting bicycling and improving bicycle safety. Several of these are discussed in this section.

The decision to use any of these treatments should be made in cooperation with local jurisdictions to ensure consistent application throughout an area. Additionally, a justification for using the treatment should be included in the project file, including any research or supporting documents justifying the use of the treatment. Use of non-standard treatments will require approval of the Resident Engineer. The headquarters Bicycle and Pedestrian Coordinator shall be consulted on the use of these treatments to ensure uniform application throughout the state.

14.2.16.4.1 Colored Bike Lanes

This treatment has obtained an Interim Approval from the FHWA for application. The interim approval assumes that the green coloring will supplement bike lane striping and marking either at conflict areas or continuously along a bike lane. Where bike lanes are designated with dotted lines (e.g., at intersections) the green paint may be continuous. Coloring of bike lanes is a supplemental treatment and should be used to emphasize the presence of properly designed bike lanes. For further information see FHWA Interim Approval for Optional Use of Green Colored Pavement for Bike Lanes (IA-14).

FHWA has developed specifications for the color.

14.2.16.4.2 Bike Boxes

A bicycle box is a designated area for bicyclists on the approach to a signalized intersection. They are located between the advance motorist stop line and the crosswalk or intersection. It is intended to provide bicyclists with a visible and safe place to wait in front of stopped motorists during the red signal phase. Designed to be used during the red phase, the box is intended to reduce car-bike conflicts, increase bicyclist visibility and provide bicyclists with a head start when the light turns green. Bike boxes allow bicyclists to group together to clear an intersection quickly, and may minimize impediments to other traffic at the onset of the green indication. Pedestrians may also benefit from reduced vehicle encroachment into the cross walk when bike boxes are present.

At intersections with high numbers of conflicts between right-turning motorists and bicyclists consideration should be given to treatments instead of or in addition to the bicycle box. These treatments may include separating conflicting traffic with a leading or exclusive signal and separating turning traffic from through traffic by providing exclusive turn lanes.

A bicycle box should be formed by placing a stop line for motor vehicles a minimum of 10 feet in advance of the crosswalk or intersection. A minimum of one bicycle symbol marking should be placed in the bicycle box. A NO TURN ON RED sign should be installed wherever a bicycle box is placed in a lane from which turns on red would otherwise be permitted.

One concern about the use of bike boxes is how conflicts are addressed when the bicyclist arrives at the intersection just as the traffic signal is turning green for motorists. The motorists are not likely to be expecting a cyclist to move left from the bike lane at the moment the light turns green. In Europe, where this treatment originates, motorists are given a yellow signal prior to the traffic signal turning green; this would serve as a warning to the approaching bicyclist. Often exclusive bicycle signals are provided for bicyclists when using the bike box treatments.

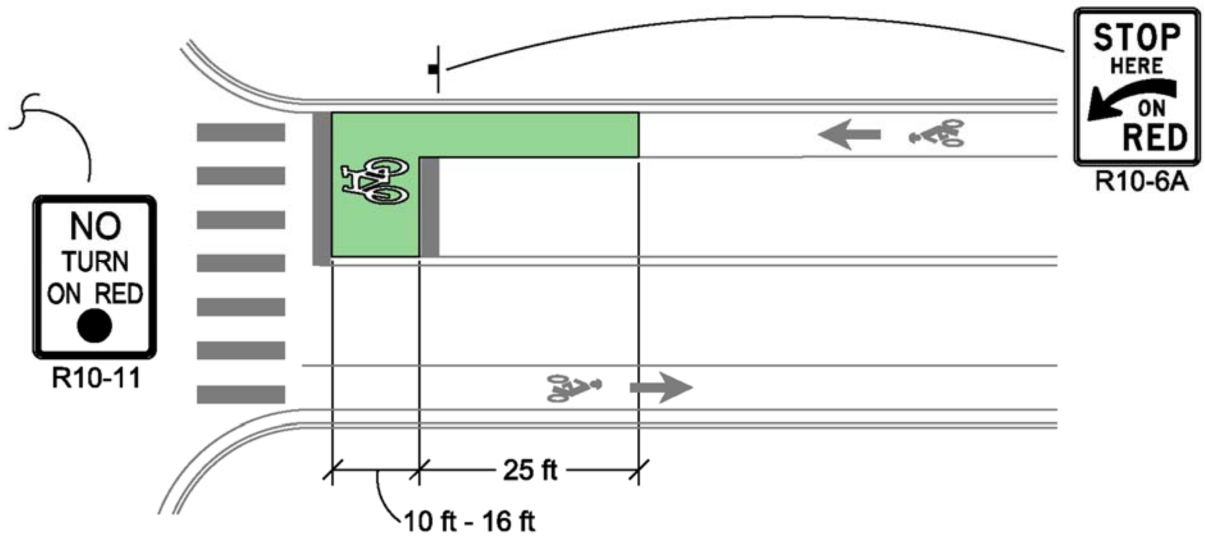


Figure 14-56 Example Striping and Marking for a Bike Box

Another operational consideration is that of right turning motorists, who are required to approach the intersection from as close to the right hand edge of the roadway as is practicable before making a right turn. In this situation motorists may block the bike lane and thus the bicyclists' access to the bike box.

A request to experiment must be submitted to FHWA prior to implementing this treatment.

14.2.16.4.3 Bicycle Signals

The *MUTCD* allows for the use of standard signal heads to control exclusive bicycle traffic movements. The use of bike specific signal heads requires the use of directional signal heads, so that bicyclists and motorists are not confused as to which signal is meant for whom. A BIKE SIGNAL (tentatively an R10-10b) sign is required to be installed immediately adjacent to every bicycle signal face that is intended to control only bicyclists.

FHWA has issued an interim approval for the use of bicycle signals (MUTCD-Interim Approval for Optional Use of a Bicycle Signal Fact (IA-16)). These signals could be used to provide a leading bicycle interval at a traffic signal, an exclusive bicycle phase, an exclusive left turn phase for bicycles on sidepaths, or as a signal for shared use paths.

The FHWA interim approval states

> *The use of a bicycle signal face is optional. However, if an agency opts to use bicycle signal faces under this Interim Approval, such use shall be limited to situations where bicycles moving on a green or yellow signal indication in a bicycle signal face are not in conflict with any simultaneous motor vehicle movement at the signalized location, including right (or left) turns on red.*

The interim approval includes signal design, mounting, and operational requirements. It is available on the internet at *http://mutcd.fhwa.dot.gov/res-interim_approvals.htm*.

14.2.16.5 Maintenance of Bicycle Facilities

Maintenance of pavement surfaces is critical to safe and comfortable bicycling. While regular maintenance activities will be required, some design treatments will help minimize maintenance needs:

- Place public utilities such as manhole covers and drainage grates outside of bikeways
- Ensure that drainage grates, if located on or near a bikeway, have narrow openings and that the grate openings are placed perpendicular to the riding surface (Figure 14-29)
- Design of appropriate cross slopes should help to keep the riding surface clear of debris and water

14.2.16.5.1 Snow and Ice Control

In designing roadways, roads should be designed to allow for snow storage. The roadside should have adequate space to place plowed snow so that it does not block a shared use path that may be adjacent to the roadway. Separation between road and path allows for snow storage.

14.3 PEDESTRIAN FACILITIES

Pedestrians and their interactions with vehicular traffic are major considerations for highway planning and design ***(1)***. Pedestrians are part of every roadway environment and they should be considered in all roadway designs. According to the *Policy on the Geometric Design of Streets and Highways (PGDSH)*:

> *Because of the demands of vehicular traffic in congested urban areas, it is often very difficult to make adequate provisions for pedestrians. Yet provisions should be made, because pedestrians are the lifeblood of our urban areas, especially the downtown and other retail areas.*

Consequently, all design projects on CDOT facilities shall include accommodations for pedestrians.

14.3.1 General Pedestrian Considerations

Pedestrian accommodations can take any of a number of forms. On CDOT projects in urban areas pedestrian accommodations will most often be represented by sidewalks. Separated shared used paths (Section 14.2 -Shared Use Paths**)** are another class of facility which may be provided for pedestrians. In rural areas, where pedestrian traffic is expected to be light, paved shoulders may accommodate pedestrians.

The degree of pedestrian accommodation provided will be influenced by the land use patterns surrounding the project, or by the planned land use patterns.

14.3.1.1 Accommodating Pedestrians in the Right-of-Way

The level of accommodation for pedestrians can be measured by a number of methods ranging from subjective to objective.

Often, as part of downtown redevelopment projects or Safe Route to School projects, a walking audit which includes subjective and objective analyses will have been performed. A walking audit documents recommended improvements to the roadway and pedestrian facilities to improve pedestrian accommodation. Any such local plans should be reviewed and the recommendations addressed in the design plans to the maximum extent feasible.

The 2010 *Highway Capacity Manual (HCM)* **(6)** establishes an objective method for determining the level of pedestrian accommodation based upon the geometric and operational characteristics of the roadway being analyzed. This method is based upon numerous research projects which quantified what factors influence how pedestrians perceive a roadway and sidewalk safety and comfort. This method is often used by agencies to set minimum target levels of accommodation for pedestrian facilities. The model for links (roadway segments between intersections) includes the following factors:

- Presence and width of a sidewalk
- Width of the outside lane
- Presence and width of a paved shoulder or bike lane
- Presence and width of a parking lane
- Percent of parking occupied by parked cars
- Presence of trees or a barrier between the sidewalk and the roadway
- Operating speeds on the roadway
- Traffic volume on the roadway.

The primary geometric conditions that are influenced by design are the presence of a sidewalk, sidewalk width, and the separation of the sidewalk from the outside lane. This *HCM* methodology is a useful tool for designing cross sectional geometry to meet a target level of pedestrian accommodation.

The *Highway Capacity Manual* also provides a method for determining the Level of Service based upon sidewalk congestion. This methodology should also be employed also to ensure adequate sidewalk width where high volumes of pedestrians are expected.

As stated above in 14.3.1 General Pedestrian Considerations, on CDOT construction projects, it is likely that sidewalks will be the facility of choice for accommodating pedestrians. However, in some cases, particularly in rural areas where traffic volumes are low and pedestrian traffic is expected to be only occasional, a paved shoulder, may be the only accommodation needed for pedestrians.

When sidewalks are included in projects, they should be continued to logical termini. For example, if a roadway project ends just prior to an intersection, pedestrian improvements should continue to the intersection.

14.3.1.2 Operating Characteristics of Pedestrians

There is no single type of design pedestrian. Pedestrians come in all sizes, and with varying degrees of physical and cognitive abilities. It is important to recognize the diversity and wide spectrum of pedestrians' abilities during facilities design.

Typical pedestrian walking speeds range from approximately 2.5 feet per second to 6.0 feet per second. The *MUTCD* states that a speed of 3.5 feet per second should be used for calculating pedestrian clearance intervals at pedestrian signals **(44)**. Such seasonal factors as ice and snow can reduce travel speeds below normal.

The space taken up by a single stationary person can be approximated by an ellipse 1.5 feet x 2 feet, with a total area of 3 square feet. In evaluating a pedestrian facility, the HCM assumes an area of 8 square feet including a buffer zone for each pedestrian **(45)**. Two pedestrians walking side by side require at least 4.7 feet of width. Two people in wheelchairs passing each other will need at least 5 feet of width, and if each has an assistive animal, 8 feet of width will be required.

According to the AASHTO Guide for the Planning, Design, and Operation of Pedestrian Facilities **(3)**,

> *In 1994, an estimated 7.4 million persons in the United States used assistive technology devices for mobility impairments, 4.6 million for orthopedic impairments, 4.5 million for hearing impairments, and 0.5 million for vision impairments. These numbers are expected to increase because there is a positive correlation between an increase in age and an increase in the prevalence rate of device usage. For example, persons who are 65 years and over use mobility, hearing, and vision assistive devices at a rate four times greater than the total population **(46)**.*

These pedestrians must be considered in the design of pedestrian facilities.

14.3.1.3 Americans with Disabilities Act Requirements

The Americans with Disabilities Act (ADA) mandates the accommodation of persons with disabilities in pedestrian facility design through the provision of *pedestrian access routes*.

A *pedestrian access route* is a continuous and unobstructed walkway within a pedestrian circulation path that provides accessibility.

The standards for accessible routes are set by the U.S. Access Board in the *Americans with Disabilities Act Architectural Guidelines for Buildings and Facilities* (*ADAAG*) **(47)**. The ADA standards for public rights-of-way, the *Public Rights-of-Way Accessibility Guidelines* (*PROWAG*) are currently in draft form **(48)**. The criteria contained within this *Roadway Design Guide* will comply with the draft *PROWAG;* notations will be made when these vary from the *ADAAG* ***(47)*.**

All newly designed and newly constructed pedestrian facilities located in the public right-of-way shall comply with these requirements. All altered portions of existing transportation facilities located in the public right-of-way shall comply with these requirements to the maximum extent feasible.

If it is technically infeasible to comply with the requirements of the ADA, documentation shall be made to the file to fully justify any non-compliant features of a design. It is not anticipated that right-of-way will be purchased for the sole purpose of complying with the ADA.

14.3.1.4 Curb Ramps and Blended Transitions

Curb ramps shall be installed where a pedestrian access route crosses a raised curb that vertically separates pedestrians from vehicles. Where sidewalks are not separated from the roadway with curb, such as on roadways with open shoulders, the at-grade connection between the sidewalk and roadway is referred to as a blended transition.

Curb ramps shall have a maximum longitudinal slope of 8.33 percent, except that the maximum required length of a curb ramp is 15 feet.

The maximum cross slope of a curb ramp is 2 percent.

A landing a minimum of 4.0 feet by 4.0 feet shall be provided at the top of the ramp run and shall be permitted to overlap other landings and clear floor or ground space. Running slope and cross slopes of landings at intersections shall be 2 percent maximum. Running and cross slope at midblock crossings shall be permitted to meet street or highway grade.

Flared sides with a slope of 10 percent maximum, measured parallel to the curb line, shall be provided where a pedestrian circulation path crosses the curb ramp. Where a curb ramp does not occupy the entire width of a sidewalk, drop-offs at diverging segments shall be protected.

The clear width of landings, blended transitions, and curb ramps, excluding flares, shall be a minimum of 4.0 feet.

Detectable warning surfaces complying with the ADAAG shall be provided where a curb ramp, landing, or blended transition connects to a street ***(47).***

Grade breaks at the top and bottom of curb ramps shall be perpendicular to the direction of the ramp run. At least one end of the bottom grade break shall be at the back of curb. Surface slopes that meet at grade breaks shall be flush.

The counter slope of the gutter or street at the foot of a curb ramp, landing, or blended transition shall be 5 percent maximum.

On a diagonal ramp, where the pedestrian is required to change direction upon entering the crosswalk, a clear space of at least 4.0 feet by 4.0 feet minimum beyond the crosswalk shall be provided within the width of the crosswalk and wholly outside the parallel vehicle travel lane.

14.3.1.5 Vertical Changes in Grade

The maximum instantaneous elevation change on a pedestrian access route without a treatment is one-quarter inch. Changes in level from one quarter to one half inch shall be beveled at a slope of no greater than 2:1. Changes in elevation greater than one half inch shall be designed with a maximum slope of 5 percent.

14.3.2 Sidewalks

Sidewalks shall be provided on all projects on CDOT facilities on which the design year land use is urban. Sidewalks should be provided on both sides of CDOT roadways on these projects.

Sidewalk surfaces shall be firm, stable, and slip resistant. Concrete sidewalks shall have a broom finish to increase skid resistance.

The pedestrian access route along a sidewalk should be designed to maximize straight through movements by pedestrians without the need to divert around utilities, street furniture, or driveways.

Adopted pedestrian plans shall be consulted to determine if a project roadway has been identified for the inclusion of pedestrian facilities. CDOT projects should implement relevant pedestrian plan facility recommendations to the maximum extent possible.

Sidewalks should also be provided on those projects where other factors indicate a need.

14.3.2.1 Separation from Roadway

The separation of a sidewalk from a roadway is an important factor in the perceived safety and comfort of a pedestrian facility ***(6)***. The greater the separation from the roadway the more pleasant the facility and consequently the more likely it is to be used by pedestrians.

Separation from the roadway provides benefits beyond the perceived safety and comfort of the pedestrian. Safety is improved by increasing separation from the roadway, particularly on roadways without curb and gutter. A buffer area provides a place to construct curb ramps and driveways outside of the sidewalk area, making it easier to comply with ADA. Buffer areas can also be used for snow storage. Utility poles, parking meters, and signs can be placed in a sufficiently wide buffer, thus ensuring the complete sidewalk width is available for pedestrians.

14.3.2.1.1 Separation from Roadway with Curb and Gutter

If a project roadway is included in an adopted pedestrian plan, the provided separation should comply with target values presented in the plan. Target values may be in the form of adopted minimum separations distances (or buffer, See Figure 14-57) or in target Level of Service values. For minimum level of service values, the separation will need to be calculated based upon roadway and traffic characteristics.

The minimum setback of a sidewalk from the back of curb to accommodate the construction of a perpendicular curb ramp outside of the sidewalk is 7.9 feet. Where possible this separation should be provided between the back of curb and sidewalk on curb and gutter projects.

The minimum width of setback to a sidewalk on an arterial roadway with curb and gutter is 6 feet. Under constrained conditions, this may be reduced to 5 feet. The minimum width of setback to a sidewalk on a local or collector roadway with curb and gutter is 4 feet. Under constrained conditions, this may be reduced to 2 feet.

Minimum separation to the sidewalk may be dictated by requirements for snow storage. Regional snow storage requirements should be considered when determining the minimum setback.

Where local jurisdictions are required to maintain the buffer and sidewalk area, maintenance agreements should be obtained during pre-construction.

14.3.2.1.2 Separation from Roadway without Curb and Gutter

If a project roadway is included in an adopted pedestrian plan, the provided separation should comply with target values presented in the plan.

Sidewalks on roadways without curb and gutter should be placed as far from the roadway as practical in the following sequence of desirability **(50)**:

1. As near the right-of-way line as possible
2. Outside of the clear zone
3. Five feet from the shoulder point
4. As far from edge of traffic lane as practical

14.3.2.2 Sidewalk Width

The minimum width for sidewalks on CDOT projects is 5 feet exclusive of the width of the curb.

Under constrained conditions the minimum width may be reduced to 4 feet exclusive of the width of the curb. This is the minimum pedestrian access route width allowed by the draft *PROWAG* ***(48)*****.** The *ADAAG* allows for a minimum accessible route of 3 feet in width **(47)**. Where less than 5 feet continuous width is provided, passing spaces shall be provided at intervals of 200 feet maximum. Passing spaces shall be a minimum of 5 feet wide for a distance of 5 feet along the sidewalk.

14.3.2.3 Protruding Objects

Protruding objects, including pedestrian amenities such as street furniture, water fountains, and informational kiosks, shall not reduce the width of the sidewalk to less than 4 feet.

Objects with leading edges more than 27 inches and not more than 80 inches above the sidewalk shall not protrude more than 4 inches into the clear pedestrian path (see Figure 14-57). Objects protruding more than 4 inches into the pedestrian path at more than 27 inches above the sidewalk may not be detectable by cane. Maintaining at least 80 inches clear to overhangs provides clear space to walk under protrusions for most pedestrians.

Objects mounted on free-standing posts or pylons, 27 inches minimum and 80 inches maximum above the sidewalk, shall not overhang into the clear pedestrian path more than 4 inches beyond the post or pylon base measured 6 inches above the sidewalk. Where a sign or other obstruction is mounted between posts or pylons and the clear distance between the posts or pylons is greater than 12 inches, the lowest edge of such sign or obstruction shall not be more than 27 inches or less than 80 inches above the sidewalk.

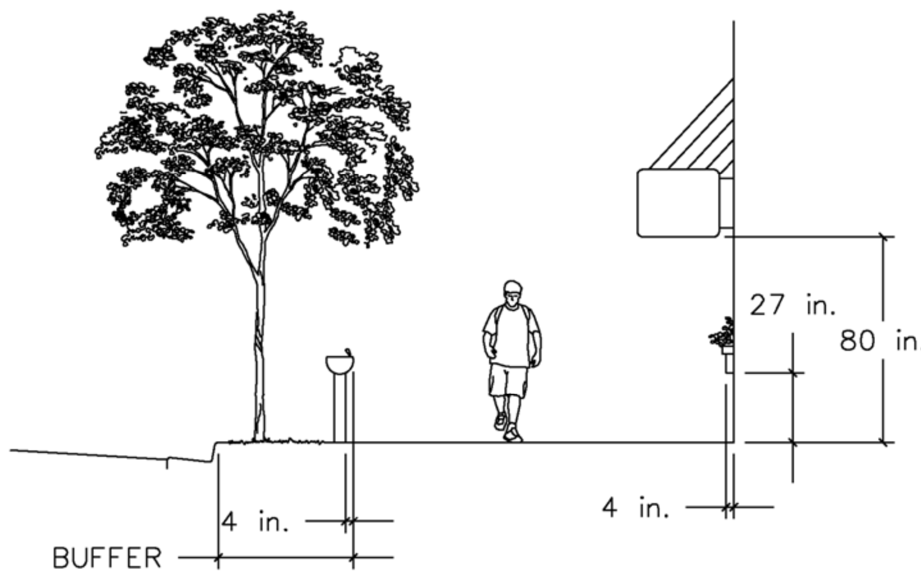


Figure 14-57 Protruding Objects

Where the vertical clearance to an obstruction is less than 80 inches, guardrails or other barriers shall be provided. The leading edge of such guardrail or barrier shall be located not more than 27 inches above the sidewalk.

14.3.3 Grade and Cross Slopes

The grade of a sidewalk should not exceed the general grade established for the adjacent street or highway.

On structures, and constructed approaches thereto, with grades exceeding 5 percent, ramps with a maximum slope of 8.33 percent and a maximum rise of 30 inches between resting intervals shall be provided. Resting intervals shall be a minimum of 5 feet measured longitudinally along the sidewalk.

The maximum cross slope for a sidewalk is 2 percent. Care must be taken so that the cross slope and longitudinal grade provide for the drainage of rain and snowmelt from the sidewalk.

14.3.4 Driveways

Where a driveway crosses a sidewalk, path of the pedestrian across the driveway must comply with the width and cross slope requirements of Section 14.3.2.2 (Sidewalk Width) and Section 14.3.3 (Grade and Cross Slopes).

14.3.5 Sidewalk Lighting

Sidewalk alignments must be considered when designing the roadway lighting pattern. Sidewalks along roadways should be lit to the same level as the adjacent roadway. This is important as pedestrians coming from the side of the road to cross must be adequately lit for motorists to see them.

Roadway lighting designed to light just the travel lanes to design levels may not provide adequate illumination for sidewalks. In these cases, supplemental lighting should be provided.

This lighting shall be consistent with requirements for walkways contained in Section 5.11 of the CDOT *Lighting Design Guide,* or as necessary, the AASHTO *Roadway Lighting Design Guide.*

14.3.6 Transit Stops

Where possible, transit waiting areas should be located outside of the sidewalk. Transit pads shall be connected to the sidewalk.

Bus stop boarding and alighting areas shall provide a clear length of 8.0 feet minimum, measured perpendicular to the curb or roadway edge, and a clear width of 5.0 feet minimum, measured parallel to the roadway.

14.3.7 Pedestrian Crossings of Roadways

Careful design of roadway crossings is critical to pedestrians' mobility and safety. Pedestrian crossings should be designed so that they are convenient for users or pedestrians will choose to cross at other locations, outside the protection of a crosswalk.

ADA compliant curb ramps or blended transitions shall be installed wherever a pedestrian access route crosses a roadway.

14.3.8 Pedestrian Crossings at Intersections

Motorists approaching intersections are primarily concerned with conflicts with other motorists. Consequently, it is important to ensure pedestrians waiting at intersections and approaching motorists are clearly visible to each other.

In urban areas, the minimum curb radii allowed for the design vehicle as found in Chapter 9 of this *Roadway Design Guide* should be used. This will reduce vehicle speeds and pedestrian crossing distances. Curb extensions should be considered to reduce crossing distances at intersections of streets with on-street parking.

14.3.8.1 Pedestrian Crossings at Uncontrolled Approaches to Intersections

Designated pedestrian crossings of uncontrolled approaches to intersections should be designed as midblock crossings. Guidance on these crossings can be found in Section 14.3.9 (Pedestrian Crossings at Midblock Locations).

14.3.8.2 Pedestrian Crossings at Stop and Yield Control Intersections

In urbanized areas, marked crosswalks should be provided wherever a sidewalk crosses a street under stop or yield control. STOP or YIELD lines shall be placed a minimum of 4 feet in advance of the crosswalks.

On multilane roadways under yield control, YIELD lines should be placed 30 feet in advance of the near edge of the intersecting roadway. This advance placement is to improve the visibility of crossing pedestrians to motorists.

14.3.8.3 Pedestrian Crossings at Signal Control Intersections

If an intersection under signal control has sidewalks, then marked crosswalks should be provided. In urbanized areas pedestrian signals are recommended at all intersections where sidewalks are provided on the approaches to a signalized intersection. STOP lines shall be placed a minimum of 4 feet in advance of the crosswalks. Consideration may be given to providing advance right turn STOP lines to improve the visibility of pedestrians coming from the motorist's left.

Pedestrian push buttons shall be accessible to pedestrians via an accessible pedestrian route in compliance with the ADA.

The draft PROWAG requires that whenever pedestrian signals are installed, accessible pedestrian push buttons be installed ***(48).***

At intersections with high volumes of right turning traffic, raised right turn channelization islands should be provided to allow pedestrians to cross the right turning traffic independently of the rest of the intersection. Single right turn channelization islands should be under yield control and have YIELD lines a minimum of 4 feet in advance of the crosswalk. Pedestrian crossings, crosswalks, and W11-2 (PEDESTRIAN CROSSING sign) should be placed on the approach end of the channelization island to maximize visibility to motorists. Space should be provided beyond the crosswalk for a single motor vehicle to store. Pedestrian signal heads for crossing of the through lanes shall be placed on the concrete channelization island.

Painted channelization islands do not provide the pedestrian advantages of raised channelization islands. Signal poles cannot be placed in painted islands. Consequently the pedestrian signal necessarily applies to the entire crossing, not just the through lanes. This precludes the use of yield control on the slip lane and the right turn must be signalized.

At multilane right turn channelization islands, the draft PROWAG requires the use of accessible pedestrian signals across the turn lanes ***(48).*** See the *MUTCD* Section 4.E.

At intersections with high volumes of pedestrians, consideration should be given to restricting the right turn on red movement. NO RIGHT ON RED blank-out signs may be used to restrict right turns only when pedestrians have pushed the pedestrian push button. This minimizes the delay to motorists due to the right turn restriction.

Additionally, YIELD TO PEDS IN CROSSWALK blank-out signs can be used to remind right-on-green and permissive left-turn movements of their obligation to yield to pedestrians in the crosswalk.

Another method to reduce pedestrian conflicts with turning motorists is through the use of a leading pedestrian interval. Where leading pedestrian intervals are used, Accessible Pedestrian Signals should be considered.

14.3.8.4 Pedestrian Crossings at Roundabouts

Research suggests that properly designed single-lane roundabouts have fewer pedestrian conflicts and crashes than typical signalized intersections **(51)**. To accommodate pedestrians, roundabouts should be designed to reduce speeds of approaching vehicles. Design speeds through single-lane roundabouts of 12 to 22 mph are typical.

Crosswalks at roundabouts shall be placed a minimum of 20 feet back from the circulating roadway. See Figure 14-58.

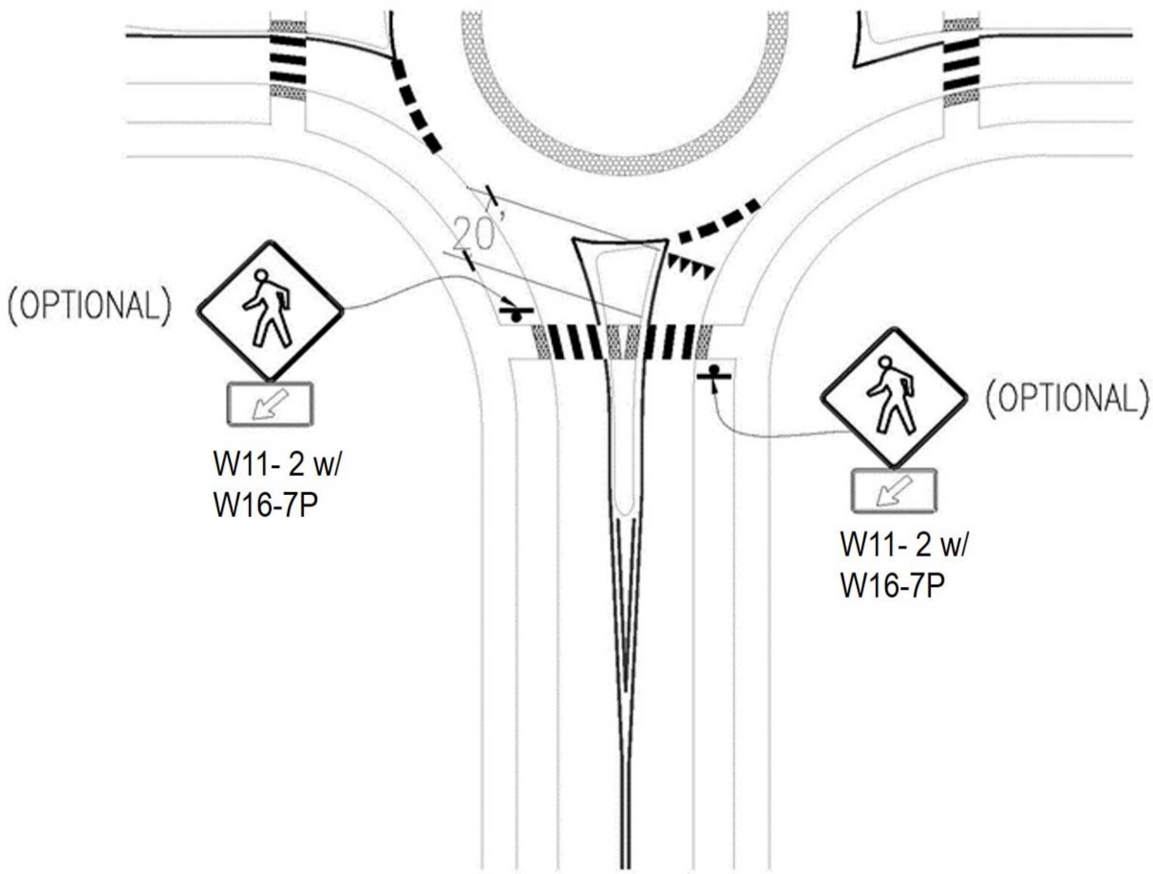


Figure 14-58 Location of Pedestrian Crossings at Roundabouts (52)

In areas prone to snow where the crosswalks may not be visible in winter, the W11-2 (PEDESTRIAN CROSSING) sign assembly should be installed the crosswalks.

The Draft PROWAG requires crosswalks across multilane approaches to roundabouts to be provided with accessible pedestrian signals ***(48).***

14.3.9 Pedestrian Crossings at Midblock Locations

When pedestrian crossing volumes meet the warrants for signalized pedestrian crossings, the installation of traffic signals for pedestrians should be considered.

The minimum clear width between crosswalk lines is 6 feet.

The *MUTCD* provides information on what type of traffic control devices may be used at midblock crossings. However, other than requiring crosswalk markings and PEDESTRIAN WARNING (W11-2) signs, it provides no clear guidance about the conditions in which any particular traffic control devices are recommended to be used to ensure motorists' yielding. The following section provides guidance in this regard. The tables provided should not be taken as requirement, rather as guidance for determining appropriate levels of traffic control at midblock crossings.

White, retroreflective crosswalk pavement markings shall be installed at all midblock crossings.

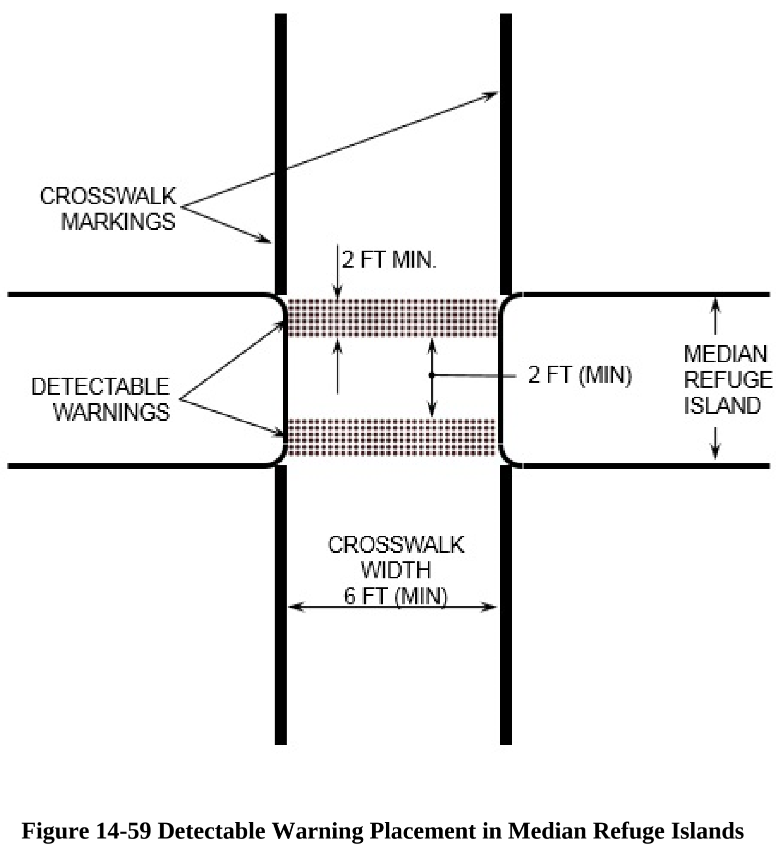


Figure 14-59 Detectable Warning Placement in Median Refuge Islands

Raised median pedestrian refuge islands should be installed at all midblock crossing locations where the pedestrian must cross four or more lanes of traffic. The minimum raised separation width between travel lanes for a pedestrian refuge island is 6 feet. For shared use path crossings the desirable minimum width of a refuge island is 10 feet. Where crossings are cut through median refuge islands detectable warnings shall be installed: two feet of detectable warnings, two feet flat surface minimum, and two feet of detectable warnings. See Figure 14-59.

Ideally, raised islands should extend along the roadway in advance of the crossing to the STOP or YIELD line.

An angled cut through of the median provides additional space for pedestrians to stage as well as encouraging them to look toward oncoming traffic. See Figure 14-60.

Advance STOP or YIELD lines shall be installed at all midblock crossing locations where the pedestrian must cross four or more lanes of traffic.

14.3.9.1 Rapid Rectangular Flashing Beacons

While not yet included in the MUTCD, RAPID RECTANGULAR FLASHING BEACONS (RRFB) have been shown to improve motorist yielding at midblock crossings. Research suggests motorist yield rates are ranging from 80 to 97 percent three years after deployment. To date this appears to be the most effective combination of traffic control devices that do not actually require the motorist to stop. This treatment has obtained an Interim Approval from the FHWA (Optional Use of the Rectangular Rapid Flashing Beacon, IA11) for application.

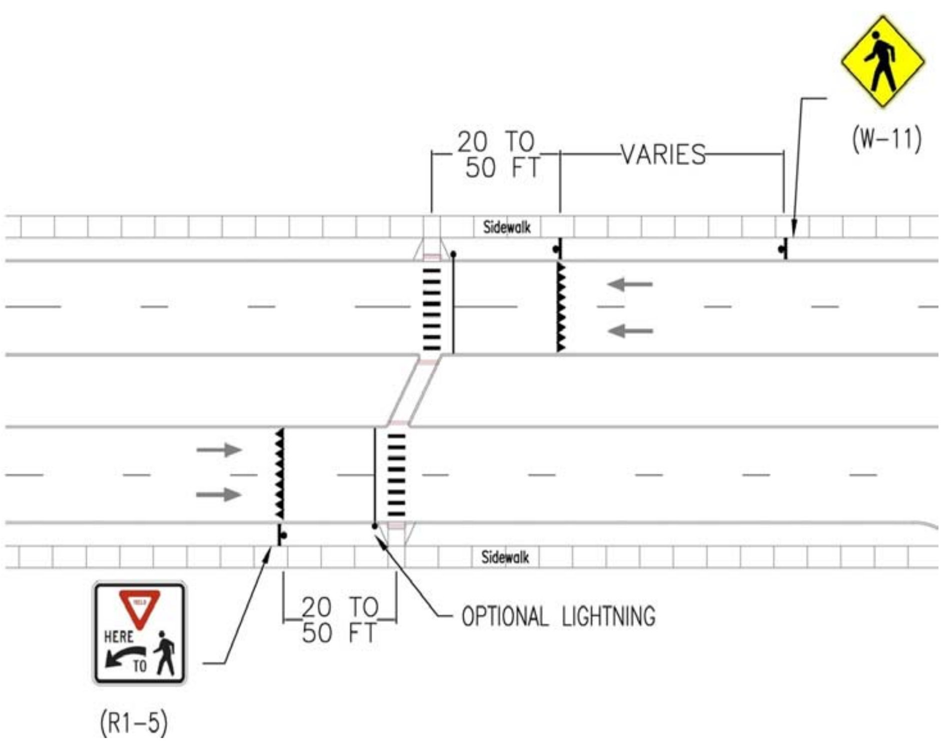


Figure 14-60 Angle Cut through a Median

The RRFB treatment is a combination of signing, markings and pedestrian activated strobe and feedback devices. Signing for the RRFB typically includes advance PEDESTRIAN WARNING signs (W11-2) with AHEAD supplemental plaques (W16-9p), and PEDESTRIAN WARNING signs (W11-2) with down arrow supplemental plaques (W16-7p). Pavement markings include yield lines and solid white lane lines (on divided multi-lane roads); the length of these lines is dependent upon the design stopping sight distance for the roadway. The pedestrian activated treatments would be the W11-2 signs with built in rectangular strobe flashers. Additionally, pedestrian visible strobes and a recorded message inform pedestrians when the crossing is activated and instruct them to wait for motorists to yield.

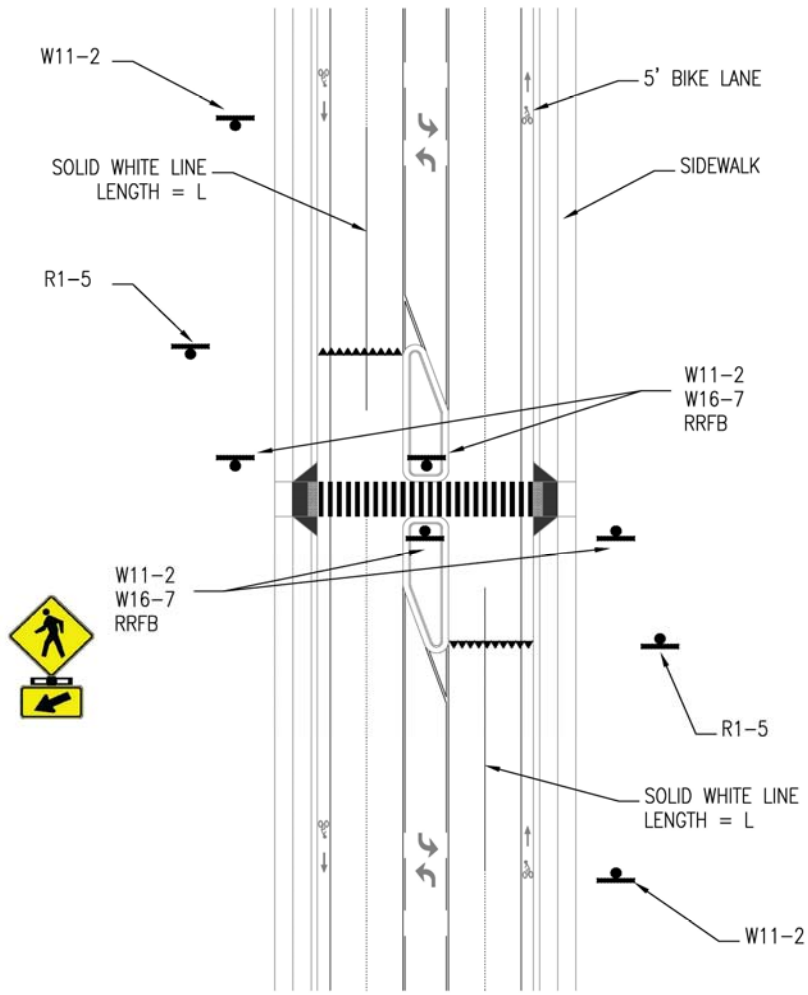
The RRFB should not be used on roadways with more than 4 through lanes. Raised medians should be provided at crossings using the RRFB to provide a space for left hand signs to be installed.

The R1-5 (YIELD HERE TO PED) shall be placed so that it does not restrict motorists' visibility of the RRFB at the crosswalk.

For the placement of advance stop lines and advance warning signs refer to the MUTCD.

High visibility crosswalks are to be used with the RRFB crossing treatment, as seen in Figure 14-61.

Timing of the flashing beacon should allow for pedestrians to scan for motorists, step from the side of the road and completely cross the street. Depending upon pedestrian volumes, 5 to 10 seconds should be provided for pedestrians to scan for gaps and enter the roadway. For areas with very high pedestrian volumes (more than 10 pedestrians per crossing), additional startup time should be provided. A minimum of 3.5 feet per second crossing speed should be assumed for pedestrians.



Speed	L
30 mph	140 feet
35 mph	185 feet
40 mph	235 feet

Figure 14-61 Rapid Rectangular Flashing Beacon

14.3.9.2 Pedestrian Hybrid Beacons

PEDESTRIAN HYBRID BEACONS are pedestrian activated beacons to warn motorists that pedestrians are crossing the street and that require the motorists to stop for pedestrians **(53)**. They do not require the satisfaction of traffic signal warrants. Chapter 4F of the MUTCD does provide some guidance regarding the volume of pedestrians crossing a roadway that would merit the consideration of a PEDESTRIAN HYBRID BEACON **(52)**.

PEDESTRIAN HYBRID BEACONS are required for use on unsignalized designated crossings of roadways with six or more lanes.

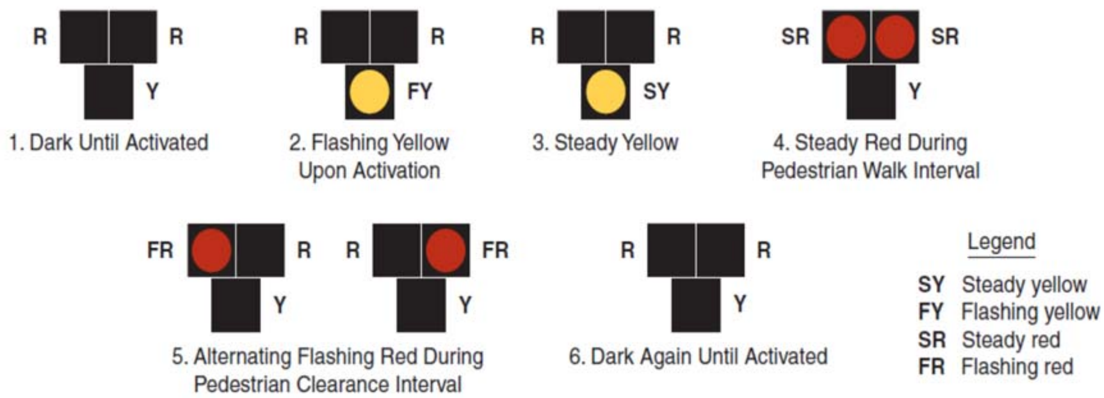


Figure 14-62 Pedestrian Hybrid Beacon Sequence (53)

The signal sequence for a pedestrian hybrid signal is shown in Figure 14-62.

14.3.9.3 Guidance for Traffic Control Selection at Midblock Crossings

For these guidelines, roadways were stratified into low-, medium-, and high-volume. The threshold volume for low- to medium-volume is determined using the amount of time a pedestrian can expect to wait for an adequate gap in traffic to cross the street. The medium- to high-volume threshold is based upon a midblock crossing safety study prepared by the University of North Carolina's Highway Safety Research Center (54). Depending on whether the street being crossed is low, medium or high volume, the corresponding value listed in

Table 14-10, would be referenced to determine the recommended traffic control devices for the crossing.

Traffic Volume in Lanes Being Crossed	Recommended Traffic Control
< 6,700 vehicles per day	Table 14-12
6,700 – 12,000 vehicles per day	Table 14-13
>12,000 vehicles per day	Table 14-14

Table 14-10 Referral Table for Midblock Crossing Treatments

Three tiers of traffic control device packages were identified for these guidelines: static signs, activated signs, and hybrid beacons. The components of each of these packages are provided in Table 14-11 below:

	Midblock Crossing Traffic Control Devices Tier		
Preferred Traffic Control Devices	Static Signs	Activated Signs	Stop Controlled
Marked Crosswalks	✓	✓	✓
Bicycle or Pedestrian Warning sign with Trail Xing Sign (W11-15) w/ (W11-15l Or Arrow (W16-7p)[2] 	✓	✓	✓
Advance Yield or Stop Lines[5]	✓	✓	✓
Trail Xing Sign (advance) and TRAIL XING Pavement Marking	✓	✓	✓
Yield or Stop Here to Ped Signs (R1-5)(R1-5)[3,4] 	✓	✓	✓
RRFB crossing Ped Xing Signs (W11-2) with rapid rectangular flashing beacons, and solid centerlines on approaches		✓	
Pedestrian Hybrid Beacon[7]			✓

Table 14-11 Traffic Control Devices Tiers

The matrices on the following pages present packages of traffic control devices recommended for specific roadway conditions. While providing guidance, there are sometimes field conditions which make the strict adherence to any typical signing and marking scheme impractical. Therefore, when applied at new locations, each location should be reviewed in the field to ensure the proposed treatments are appropriate.

If sight distance is limited, additional traffic control may be appropriate.

Additional traffic control may be appropriate in areas where expected pedestrians are predominately school children or individuals with mobility impairments.

The following general notes should be considered when using Table 14-12, Table 14-13, and Table 14-14.

	Lanes	2 lanes						4 lanes					
	Median	No			Yes			No			Yes		
	Speed	≤ 30 mph	35–40 mph	≥ 45 mph	≤ 30 mph	35–40 mph	≥ 45 mph	≤ 30 mph	35–40 mph	≥ 45 mph	≤ 30 mph	35–40 mph	≥ 45 mph
Taffic Control Devises Package	Static Signs	✓			✓	✓		✓			✓		
	Rectangular Rapid Flashing Beacon		✓	✓			✓		✓	✓		✓	✓
	Hybrid Beacon												

Table 14-12 Roadway Volume less than 650 Vehicles per hour, vph (6,700 vehicles per day[1], vpd)

	Lanes	2 lanes						4 lanes						6 lanes					
	Median	No			Yes			No			*Yes*			No			Yes		
	Speed	≤ 30 mph	35–40 mph	≥ 45 mph	≤ 30 mph	35–40 mph	≥ 45 mph	≤ 30 mph	35–40 mph	≥ 45 mph	≤ 30 mph	35–40 mph	≥ 45 mph	≤ 30 mph	35–40 mph	≥ 45 mph	≤ 30 mph	35–40 mph	≥ 45 mph
Taffic Control Devises Package	Static Signs	✓			✓						✓								
	Rectangular Rapid Flashing Beacon		✓	✓		✓	✓	✓				✓	✓						
	Hybrid Beacon								✓	✓				✓	✓	✓	✓	✓	✓

Table 14-13 Roadway Volume greater than 650 vph[1] (6,700 vpd), and less than 1,150 vph (12,000 vpd)

	Lanes	2 lanes						4 lanes						6 lanes					
	Median	No			Yes			No			*Yes*			No			Yes		
	Speed	≤ 30 mph	35–40 mph	≥ 45 mph	≤ 30 mph	35–40 mph	≥ 45 mph	≤ 30 mph	35–40 mph	≥ 45 mph	≤ 30 mph	35–40 mph	≥ 45 mph	≤ 30 mph	35–40 mph	≥ 45 mph	≤ 30 mph	35–40 mph	≥ 45 mph
Taffic Control Devises Package	Static Signs				✓														
	Rectangular Rapid Flashing Beacon	✓	✓	✓		✓	✓	✓			✓	✓	✓						
	Hybrid Beacon								✓	✓				✓	✓	✓	✓	✓	✓

Table 14-14 Roadway Volume greater than 1,150[1] vph (12,000 vpd)

General notes for applying the Crossing Treatment Guidelines Matrices:

Each column in the table represents a package of traffic control devices recommended for the specific crossing condition.

Volumes in the title cells assume a daily to peak hour volume factor of 0.97.

The designation of "YES" for the median assumes there is potential for installing a raised median at the crossing location and that one will be installed. Raised medians that can be used as pedestrian refuges (6 feet wide or wider in the direction of the roadway cross-section) will allow for less restrictive motor vehicle traffic controls to be used in conjunction with the midblock crossings. Wider refuge islands, 10 feet or more, should be considered to accommodate bicycle with trailers and recumbent bicycles.

On roadways with two-way left turn lanes, refuge islands should be installed at crossing locations.

On multi-lane roadways with medians on the approach, crossing signs for motorists should be placed in the medians as well as on the side of the roadway.

The use of angled cuts through the median (sometimes referred to as Danish offsets) should be considered at all crossings with raised medians for two reasons. First, the offset through the median directs the path users' attention toward the traffic about to be crossed. Secondly, of particular importance when using these tables for shared use path intersections, by providing an angled cut through the median, longer users (tandems, bicycles with trailers) may be better accommodated than in a narrower median.

When advance yield lines are used on the approach roadways they should be used in conjunction with solid lane lines. The lane lines should extend a distance equal to the stopping sight distance back from the yield lines. This is to enable law enforcement officers to determine when a motorist fails to yield when he could have done so.

On six-lane, undivided roadways, strong consideration should be given to providing a signalized crossing of the roadway for pedestrians. Until such time as this can be achieved, aggressive channelization should be used to divert pathway users to the nearest safe crossing.

This guidance assumes that lighting will be provided for crossings to be used at night.

14.3.9.4 Additional Treatments at Midblock Crossings

On roadways with on street parking, mid-block curb extensions should be considered to reduce pedestrian crossing distances. Curb extensions also improve pedestrian and motorist sight lines. Drainage must be addressed when designing curb extensions.

On lower speed and volume arterials and collector streets raised crosswalks may be considered. Raised crosswalks decrease motorist speeds, resulting in greater yielding rates. Snow plow operators have reported problems plowing over raised crosswalks; the use of short vertical curves instead of grade break lines may address this operational problem. Drainage must be addressed when designing raised crosswalks.

The approach slopes for raised crosswalks shall be marked in accordance with the *MUTCD* required markings for raised pedestrian **(54)** crossings as shown Figure 14-63.

14.3.9.5 Signalized Pedestrian Crossings

Where signal warrants for pedestrian crossings are met, the installation of traffic signals should be considered. At midblock locations accessible pedestrian signals shall be provided.

Where accessible pedestrian signals are to be installed, they shall comply with all the requirements of the *MUTCD*.

14.3.9.6 Grade Separated Pedestrian Crossings

In some locations a grade separated crossing will be the only practical method of getting pedestrians across a roadway. Common examples include crossings of expressways and where children must cross major arterials. When appropriately designed, grade separated pedestrian crossings improve the mobility and safety of pedestrians. Attributes of such a pedestrian crossing include the following ***(3):***

- The facility must be located where it is needed and will actually be used.
- Crossing structures must be built with adequate widths based on perceptions of safety as well as pedestrian volumes.
- The design must be accessible for all users.
- Barriers and railings must be provided to add an increased sense of safety to the pedestrian.
- The facility must be lit to provide an increased level of security to the pedestrian.

Where grade separated crossings are installed, approaches must meet grade criteria provided in Section 14.3.3 Grade and Cross Slopes.

Where the designer has a choice between a tunnel and an overpass, an overpass is often preferable. Overpasses have security advantages. Additionally, lighting is often a requirement for tunnels and may not be necessary for an overpass. Drainage may also be easier to accommodate on overpasses. Underpasses are often more difficult to construct because of utility conflicts or phasing issues. Additionally, pedestrians are more likely to use an overpass than an underpass. However, overpasses have significantly greater vertical clearance requirements, 17 feet 6 inches over the roadway as opposed to 10 feet over the path surface.

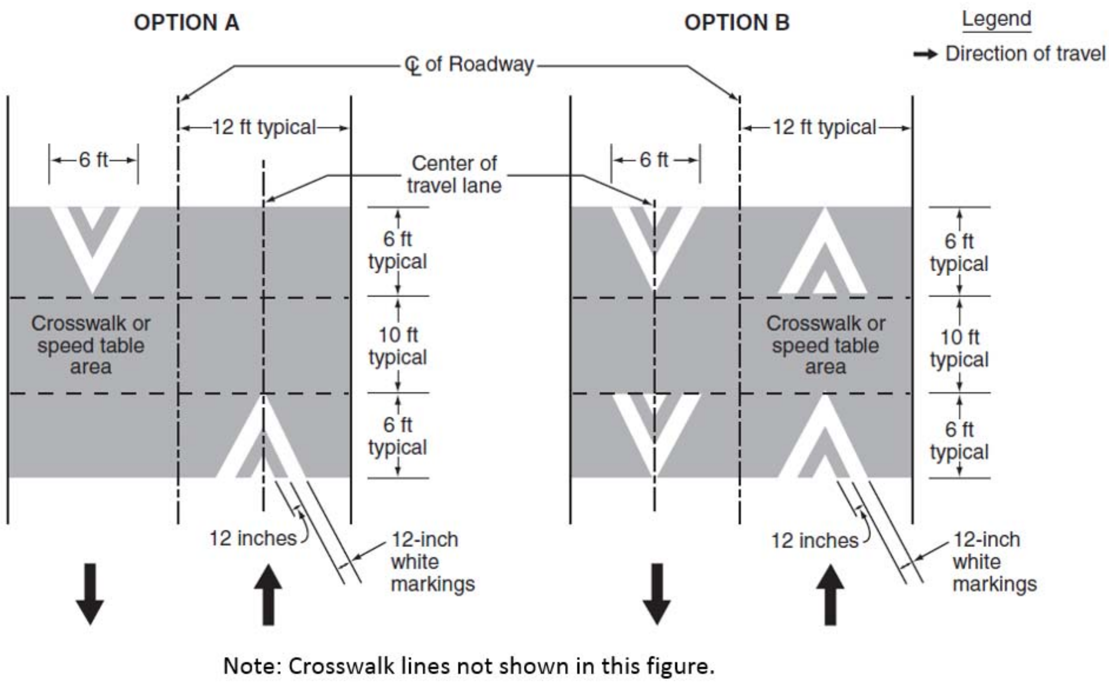


Figure 14-63 Approach Slope Markings for Raised Pedestrian Crossings (55)

When considering a grade separated pedestrian crossing a feasibility study shall be conducted. This study shall quantify current and future pedestrian use, as well as alternatives for at-grade crossings.

Contrasting crosswalk coloring is often requested in downtown areas. The use of contrasting coloring does not eliminate the requirement to mark crosswalks with white, retroreflective pavement markings.

14.3.9.7 Sidewalk Crossings of Rail Lines

Where sidewalks cross rail road tracks, appropriate crossing treatments shall be provided.

Of particular importance to individuals with mobility impairments is the interface between the rails and the sidewalk. Sidewalk surfaces shall be flush with the tops of rails. Openings for wheel flanges at pedestrian crossings of freight rail track shall be 3 inches maximum. Openings for wheel flanges at pedestrian crossings of non-freight rail track shall be 2.5 inches maximum.

Detectable warnings shall be placed on the approaches to all rail crossings unless the rail crossing is included within a roadway crossing. The detectable warning surface shall be located so that the edge nearest the rail crossing is 6 feet minimum and 15 feet maximum from the centerline of the nearest rail. The rows of truncated domes in a detectable warning surface shall be aligned to be parallel with the direction of wheelchair travel.

When used at Light Rail Transit (LRT) crossings, pedestrian signal heads shall comply with the provisions of the *MUTCD* **(56)**.

Where a sidewalk crosses a light rail transit line, Flashing-light signals (see Figure 14-64) with a CROSSBUCK (R15-1) sign and an audible device should be installed at pedestrian crossings where an engineering study has determined that the sight distance is not sufficient for pedestrians and bicyclists to complete their crossing prior to the arrival of the LRT traffic at the crossing, or where LRT speeds exceed 35 mph.

If an engineering study shows that flashing-light signals with a CROSSBUCK sign and an audible device would not provide sufficient notice of approaching light rail transit traffic, the LOOK (R15-8) sign, pedestrian gates, or both, should be considered.

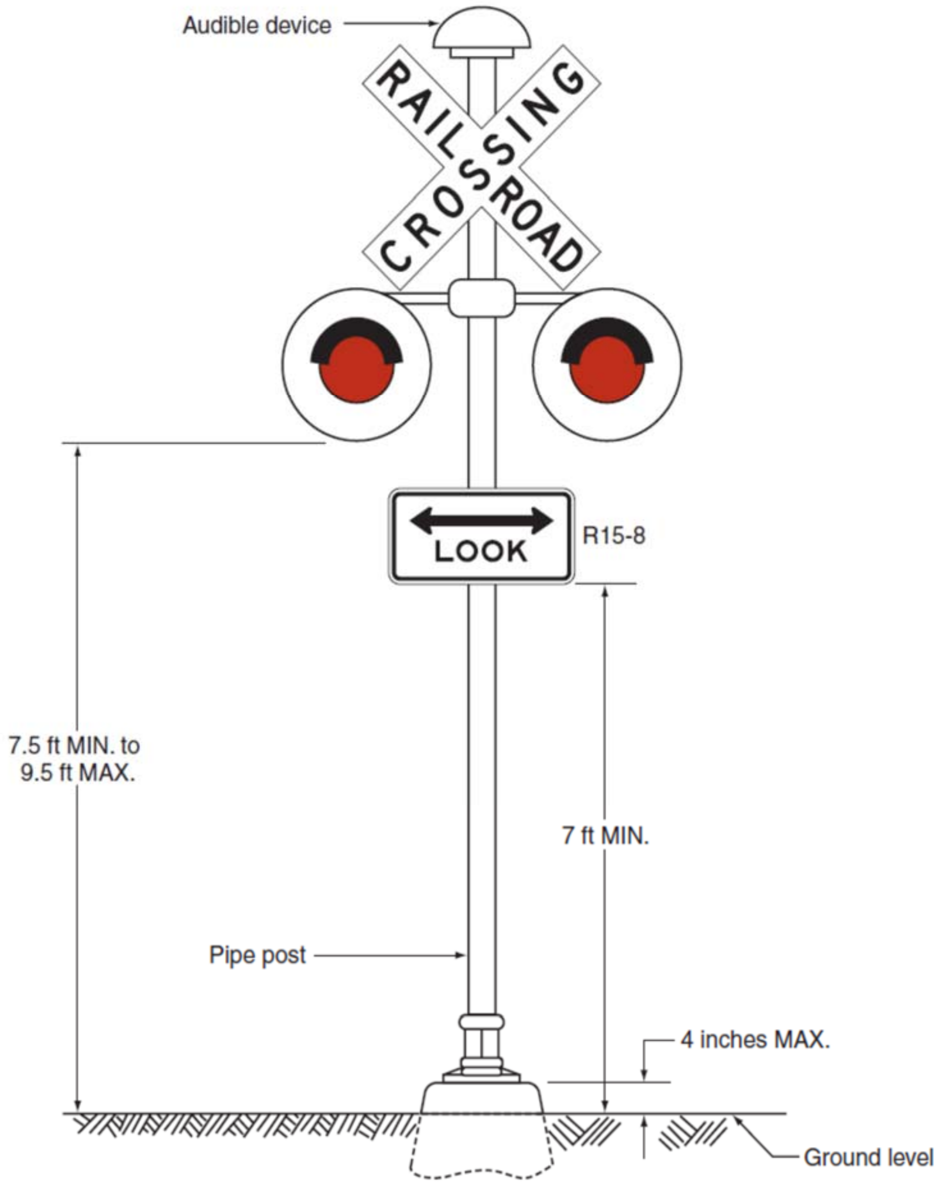


Figure 14-64 Example of Flashing-Light Signal Assembly for Pedestrian Crossings (56)

14.3.10 Other Pedestrian Considerations

14.3.10.1 Traffic Calming

The Institute of Transportation Engineers (ITE) defines traffic calming as follows:

> *Traffic calming is the combination of mainly physical measures that reduce the negative effects of motor vehicle use, alter driver behavior and improve conditions for non-motorized street users.* ***(57)***

Traffic calming differs from the application of traffic control devices in that they use roadway geometrics rather than enforcement to compel people to drive more slowly. Vertical and horizontal alignment are used to physically restrict the speeds motorists are comfortable driving. Thus, traffic calming is self-enforcing.

Traffic calming is often used in combination with other treatments such as landscaping and lighting. While these additional treatments do not compel drivers to slow down, they may provide a visual signal to drive more slowly.

Traffic calming is popular in many communities because it is effective when applied properly. By reducing speeds, the number of traffic crashes is reduced and those crashes that do occur are often less severe than on uncalmed streets. By reducing speeds, pedestrians' perceptions of safety and comfort are improved as well.

ITE and FHWA have produced the document *Traffic Calming: State of the Practice* **(58)** for informational purposes. While it does not include recommendations on the best course of action or the preferred application of the data, it does provide an excellent resource for those considering the application of traffic calming treatments.

14.3.10.2 Pedestrian Amenities

Pedestrian amenities can provide a more pleasant walking environment and thus encourage more pedestrian activity. Pedestrian amenities can include aesthetic paving treatments, street furniture, shade trees, enhanced lighting, landscaping, informational signing, and public art. Because transit users begin and end their trips as pedestrians, amenities - particularly street furniture and informational signing - can encourage greater transit use. Prior to installing pedestrian amenities, a maintenance agreement should be in place to ensure local jurisdictions the amenities will be maintained.

If aesthetic paving treatments are used they shall be firm, stable, and slip resistant. Cobbles or other treatments that create a vibratory surface for wheelchair users shall not be used within the pedestrian walkway; they may be used in border areas.

Pedestrian amenities shall be designed so that they do not reduce the pedestrian access route to less than 4 feet and shall meet all the criteria of Section 14.3.2.3 Protruding Objects.

Shade trees and landscaping shall be designed so as not to restrict intersection sight distances, or to restrict pedestrian or motorists sight distances at midblock crossings.

14.3.10.3 Pedestrian Wayfinding Signing

Pedestrian wayfinding signing is important to provide information on walk routes to destinations and attractions for pedestrians. Pedestrian wayfinding can encourage pedestrian activity and transit use.

Specific pedestrian routes can be developed. The development of pedestrian routes should include the participation of local agencies and walking interest groups.

The *MUTCD* does not provide specific signs to be used for pedestrian wayfinding. Local jurisdictions may be consulted concerning the design or visual theme of pedestrian signage.

However, standard alphabets with a minimum text height of 2 inches shall be used for pedestrian signs to ensure legibility.

14.3.10.4 On-street Parking

The presence of on-street parking significantly impacts the pedestrian environment. On-street parking provides an additional buffer between the travel lanes and the sidewalk; thus, it improves pedestrians' perceptions of safety and comfort. On-street parking often results in reduced motor vehicle travel speeds, further improving the pedestrian environment. By its very nature, on-street parking encourages pedestrian activity, walking along the road and increasing the number of pedestrians crossing the street.

Where on-street parking exists, curb extensions should be considered to restrict parking near intersections and midblock crossing locations. Drainage patterns will need to be considered during the design of curb extensions.

14.3.11 School Areas

School zones represent a particular challenge to pedestrian design. Children are the most unpredictable, least traffic savvy of pedestrians.

Special consideration should be given to designing pedestrian facilities near schools. Sidewalks should be located as far from the roadway as possible. In some locations, it may be advisable to channelize school children with fences or other barriers; such barriers should be designed so that they do not create sight distance limitations.

If midblock crossings are installed for school crossings, enhanced treatments shall be considered. Roadway volume thresholds for

Table 14-11, Table 14-12, Table 14-13 should be reduced by 20 percent. School children shall not be required to cross more than two lanes without a traffic signal. On roadways with raised pedestrian refuge islands, a four-lane divided roadway is the maximum width crossing without a traffic signal that may be provided specifically for school children.

Reduced speed zones may be considered in school zones. When using the SCHOOL SPEED LIMIT ASSEMBLY, the use of timed flashers is recommended (Figure 14-65). The use of the WHEN CHILDREN PRESENT (S4-3) plaque is not recommended.

Figure 14-65 SCHOOL SPEED LIMIT Assembly

Consideration should be given to restricting right turn on red during periods when students are walking to and from school. Again, use of the WHEN CHILDREN PRESENT (S4-3) plaque is not recommended. Consideration should be given to using designated times for the no right on red or using blank-out signs pre-timed to school walking periods.

Pedestrian staging areas at intersections and midblock crossings should be designed to accommodate the expected volume of students who will be waiting to cross.

Student drop-off and pickup areas should be contained within the school site. If this is not possible and street-side drop-off and pickup is allowed, it shall not require students to make an unsupervised crossing of a roadway.

14.3.12 Maintenance of Traffic (58)

The following section is taken from the *MUTCD*. It includes the guidance and standard sections form the *MUTCD*. For support text, see section 6D of the *MUTCD*.

14.3.12.1 Pedestrian Considerations in Temporary Traffic Control Zones

Advance notification of sidewalk closures shall be provided by the maintaining agency or contractor.

If the temporary traffic control (TTC) zone affects the movement of pedestrians, adequate pedestrian access and walkways shall be provided. If the TTC zone affects an accessible and detectable pedestrian facility, the accessibility and detectability shall be maintained along the alternate pedestrian route.

The following three items should be considered when planning for pedestrians in TTC zones:

- Pedestrians should not be led into conflicts with vehicles, equipment, and operations

- Pedestrians should not be led into conflicts with vehicles moving through or around the worksite
- Pedestrians should be provided with a convenient and accessible path that replicates as nearly as practical the most desirable characteristics of the existing sidewalks or footpaths

A pedestrian route should not be severed or moved for non-construction activities such as parking for vehicles and equipment.

To accommodate the needs of pedestrians, including those with disabilities, the following considerations should be addressed when temporary pedestrian pathways in TTC zones are designed or modified:

- Provisions for continuity of accessible paths for pedestrians should be incorporated into the TTC plan.
- Access to transit stops should be maintained.
- A smooth, continuous hard surface should be provided throughout the entire length of the temporary pedestrian facility. There should be no curbs or abrupt changes in grade or terrain that could cause tripping or be a barrier to wheelchair use. The geometry and alignment of the facility should meet the applicable requirements of the "Americans with Disabilities Act Accessibility Guidelines for Buildings and Facilities (ADAAG)" **(48)**.
- The width of the existing pedestrian facility should be provided for the temporary facility if practical.

Traffic control devices and other construction materials and features should not intrude into the usable width of the sidewalk, temporary pathway, or other pedestrian facility. When it is not possible to maintain a minimum width of 60 inches throughout the entire length of the pedestrian pathway, a 60 x 60-inch passing space should be provided at least every 200 feet to allow individuals in wheelchairs to pass.

Blocked routes, alternate crossings, and sign and signal information should be communicated to pedestrians with visual disabilities by providing devices such as audible information devices, accessible pedestrian signals, or barriers and channelizing devices that are detectable to the pedestrians traveling with the aid of a long cane or who have low vision. Where pedestrian traffic is detoured to a TTC signal, engineering judgment should be used to determine if pedestrian signals or accessible pedestrian signals should be considered for crossings along an alternate route.

When channelization is used to delineate a pedestrian pathway, a continuous detectable edging should be provided throughout the length of the facility such that pedestrians using a long cane can follow it. These detectable edgings should comply with the provisions of the MUTCD.

Signs and other devices mounted lower than 7 feet above the temporary pedestrian pathway should not project more than 4 inches into accessible pedestrian facilities.

Fencing should not create sight distance restrictions for road users. Fences should not be constructed of materials that would be hazardous if impacted by vehicles. Wooden railing, fencing, and similar systems placed immediately adjacent to motor vehicle traffic should not be used as substitutes for crashworthy temporary traffic barriers.

Ballast for TTC devices should be kept to the minimum amount needed and should be mounted low to prevent penetration of the vehicle windshield.

Movement by work vehicles and equipment across designated pedestrian paths should be minimized and, when necessary, should be controlled by flaggers or TTC. Staging or stopping of work vehicles or equipment along the side of pedestrian paths should be avoided, since it encourages movement of workers, equipment, and materials across the pedestrian path.

Access to the work space by workers and equipment across pedestrian walkways should be minimized because the access often creates unacceptable changes in grade, and rough or muddy terrain, and pedestrians will tend to avoid these areas by attempting non-intersection crossings where no curb ramps are available.

A canopied walkway may be used to protect pedestrians from falling debris, and to provide a covered passage for pedestrians. Covered walkways should be sound construction and adequately lighted for nighttime use.

When pedestrian and vehicle paths are rerouted to a closer proximity to each other, consideration should be given to separating them by a temporary traffic barrier. If a temporary traffic barrier is used to shield pedestrians, it should be designed to accommodate the specific site conditions. Guidance for locating and designing temporary traffic barriers can be found in Chapter 9 of AASHTO's *Roadside Design Guide*.

Short intermittent segments of temporary traffic barrier shall not be used because they nullify the containment and redirective capabilities of the temporary traffic barrier, increase the potential for serious injury both to vehicle occupants and pedestrians, and encourage the presence of blunt, leading ends. All upstream leading ends that are present shall be appropriately flared or protected with properly installed and maintained crashworthy cushions. Adjacent temporary traffic barrier segments shall be properly connected in order to provide the overall strength required for the temporary traffic barrier to perform properly.

Normal vertical curbing shall not be used as a substitute for temporary traffic barriers when temporary traffic barriers are needed.

If a significant potential exists for vehicle incursions into the pedestrian path, pedestrians should be rerouted (see Figure 14-66) or temporary traffic barriers should be installed.

Tape, rope, or plastic chain strung between devices are not detectable, do not comply with the design standards in the "Americans with Disabilities Act Accessibility Guidelines for Buildings and Facilities (ADAAG)", and should not be used as a control for pedestrian movements (47).

In general, pedestrian routes should be preserved in urban and commercial suburban areas. Alternative routing should be discouraged.

The highway agency in charge of the TTC zone should regularly inspect the activity area so that effective pedestrian TTC is maintained.

Figure 14-66 Pedestrian Facility DETOUR Sign

14.3.12.2 Accessibility Considerations

The extent of pedestrian needs should be determined through engineering judgment or by the individual responsible for each TTC zone situation. Adequate provisions should be made for pedestrians with disabilities.

When existing pedestrian facilities are disrupted, closed, or relocated in a TTC zone, the temporary facilities shall be detectable and include accessibility features consistent with the features present in the existing pedestrian facility. Where pedestrians with visual disabilities normally use the closed sidewalk, a barrier that is detectable by a person with a visual disability traveling with the aid of a long cane shall be placed across the full width of the closed sidewalk.

If a pushbutton is used to provide equivalent TTC information to pedestrians with visual disabilities, the pushbutton should be equipped with a locator tone to notify pedestrians with visual disabilities that a special accommodation is available, and to help them locate the pushbutton.

REFERENCES

1. AASHTO. *A Policy on Geometric Design of Highways and Streets*, American Association of State Highway and Transportation Officials, Washington, D.C.: 2004.
2. AASHTO. *Guide for the Development of Bicycle Facilities,* American Association of State Highway and Transportation Officials, Washington, D.C.: 1999.
3. AASHTO. *Guide for the Planning, Design, and Operation of Pedestrian Facilities*, American Association of State Highway and Transportation Officials, Washington, D.C.: 2004.
4. FHWA. *Manual on Uniform Traffic Control Devices (MUTCD),* Federal Highway Administration, Washington, D.C.: 2009.
5. Colorado Revised Statutes, Section 42-1-102(10).
6. TRB, *Highway Capacity Manual 2010*, Transportation Research Board, Washington, D.C.:2010. This cite assumes HCM adoption prior to the adoption of this chapter.
7. Landis, B.W, Petritsch, T.A and Huang, H.F. *Characteristics of Emerging Road and Trail Users and Their Safety. Washington*, DC: Federal Highway Administration, 2004. FHWA-HRT-04-104.
8. FHWA. *MUTCD*, Section 9B.21, FHWA. Manual on Uniform Traffic Control Devices, Federal Highway Administration, Washington, D.C.: 2009.
9. FHWA. *MUTCD*, Section 9B.06, FHWA. Manual on Uniform Traffic Control Devices, Federal Highway Administration, Washington, D.C.: 2009.
10. Colorado Revised Statutes, Section 42-4-1412(5).
11. FDOT, *Florida Traffic Engineering Manual*, Florida Department of Transportation, Tallahassee, FL: 2010.
12. FHWA. *MUTCD*, Section 9C.07, FHWA. Manual on Uniform Traffic Control Devices, Federal Highway Administration, Washington, D.C.: 2009.
13. AASHTO. *Guide for the Development of Bicycle* Facilities, pp. 16, American Association of State Highway and Transportation Officials, Washington, D.C.: 1999.
14. Colorado Revised Statutes, 42-4-901(1)(a).
15. FHWA. *MUTCD*, Section 3A.06, FHWA. Manual on Uniform Traffic Control Devices, Federal Highway Administration, Washington, D.C.: 2009.
16. Stutts, J.C and Hunter, W.W. *Injuries to Pedestrians and Bicyclists: An Analysis Based on Hospital Emergency Department Data*. Washington, DC : Federal Highway Administration, 1997. FHWA-RD-99-10 078.

17. Hunter, W.W, Stutts, J.C, Pein, W.E and Cox, C.L. *Pedestrian and Bicycle Crash Types of the Early 1990's*. Washington, DC : Federal Highway Administration, 1996. FHWA-RD-95-163.

18. Wachtel and Lewiston. "Risk Factors for Bicycle-Motor Vehicle Collisions at Intersections." ITE Journal, September, 1994.

19. Moritz, W. "Adult bicyclists in the United States - Characteristics and Riding Experience in 1996." *Transportation Research Record 1636*, TRB, National Research Council, Washington, D.C.: 1998

20. Petritsch, Landis, Huang, and Challa. "Sidepath Safety Model - Bicycle Sidepath Design Factors Affecting Crash Rates." *Transportation Research Record 1982*, National Research Council, Washington D.C.: 2006

21. Colorado Revised Statutes, Section 42-4-1007(1)(a).

22. FHWA. *MUTCD*, Section 9C.04, FHWA. Manual on Uniform Traffic Control Devices, Federal Highway Administration, Washington, D.C.: 2015.

23. Colorado Revised Statutes, Section 42-4-1412(8).

24. SRF Consulting Group, Inc., "Bicycle and Pedestrian Transportation" FHWA, Washington, D.C.: 2003.

25. FHWA. *MUTCD*, Section 9C.05, FHWA. Manual on Uniform Traffic Control Devices, Federal Highway Administration, Washington, D.C.: 2009.

26. FHWA. *MUTCD*, Section 9B.13, FHWA. Manual on Uniform Traffic Control Devices, Federal Highway Administration, Washington, D.C.: 2009.

27. FHWA. *MUTCD*, Section 9D.02, FHWA. Manual on Uniform Traffic Control Devices, Federal Highway Administration, Washington, D.C.: 2009.

28. FHWA, *Traffic Signal Timing Manual*, FHWA ,Washington, D.C.: 2008.

29. FHWA "Accommodating Bicycle and Pedestrian Travel: A Recommended Approach – A US DOT Policy Statement on Integrating Bicycling and Walking into Transportation Infrastructure," 2000.

30. Colorado Revised Statutes, Section 42-4-204.

31. FHWA. *Shared Use Path Level of Service - A User's Guide,* FHWA-HRT-05-138, Federal Highway Administration, Washington, D.C.: 2006.

32. FHWA. *Shared Use Path Level of Service Calculator*, available online at http://www.tfhrc.gov/safety/pedbike/pubs/05138/ , FHWA-HRT-05-138, Federal Highway Administration, Washington, D.C.: 2006.

33. FHWA. *MUTCD*, Section 9B, FHWA. Manual on Uniform Traffic Control Devices, Federal Highway Administration, Washington, D.C.: 2009.

34. Architectural Barriers Act Accessibility Guidelines for Outdoor Developed Areas. United States Access Board, Washington, D.C.: 2007.

35. FHWA. *MUTCD*, Section 9B.03, FHWA. Manual on Uniform Traffic Control Devices, Federal Highway Administration, Washington, D.C.: 2009.

36. FHWA. *MUTCD*, Section 4C, FHWA. Manual on Uniform Traffic Control Devices, Federal Highway Administration, Washington, D.C.: 2009.

37. FHWA. *MUTCD*, Section 9B.26, FHWA. Manual on Uniform Traffic Control Devices, Federal Highway Administration, Washington, D.C.: 2009.

38. USDA, *Equestrian Design Guidebook for Trails, Trailheads, and Campgrounds,* US Department of Agriculture (in cooperation with FHWA), Washington, D.C.: 2007.

39. FHWA. *MUTCD*, Section 6, FHWA. Manual on Uniform Traffic Control Devices, Federal Highway Administration, Washington, D.C.: 2009.

40. APBP, *Bicycle Parking Guidelines*, 2nd Edition, Association of Bicycle and Pedestrian Professionals, Cedarburg, WI: 2010.

41. Adapted from City of Toronto, DRAFT Guidelines for the Design and Management of Bicycle Parking Facilities, City of Toronto: 2008.

42. MN/DOT, MN/DOT Bikeway Facility Design Manual, St. Paul, MN: 2007.

43. Oregon Department of Transportation, Oregon Bicycle and Pedestrian Plan, Salem, OR: 1995.

44. FHWA. *MUTCD*, Section 4E.06, FHWA. Manual on Uniform Traffic Control Devices, Federal Highway Administration, Washington, D.C.: 2009.

45. TRB, *Highway Capacity Manual 2000*, Transportation Research Board, Washington, D.C.:20090.

46. U.S. Department of Health and Human Services, Centers for Disease Control and Prevention, National Center for Health Statistics. Trends and Differential Use of Assistive Technology Devices: United States, 1994. *Advance Data*, No. 292, November 13, 1997.

47. U.S. Access Board, Americans with Disabilities Act Architectural Guidelines for Buildings and Facilities, U.S. Access Board, Washington, D.C:2002. (ADAAG)

48. U.S. Access Board, *Revised Draft Guidelines for Accessible Public Rights-of-Way,* U.S. Access Board, Washington, D.C.: 2005. (PROWAG)

49. FHWA. *MUTCD*, Section 8D.03, FHWA. Manual on Uniform Traffic Control Devices, Federal Highway Administration, Washington, D.C.: 2009.

50. FDOT, *Plans Preparation Manual*, Florida Department of Transportation, Tallahassee, FL: 2010.

51. FHWA, *Roundabouts: An Informational Guide*, Report FHWA-RD-00-067, Federal Highway Administration, Washington, D.C.:2000.

52. FHWA. *MUTCD*, Section 3C.01, FHWA. Manual on Uniform Traffic Control Devices, Federal Highway Administration, Washington, D.C.: 2009.

53. FHWA. *MUTCD*, Section 4F, FHWA. Manual on Uniform Traffic Control Devices, Federal Highway Administration, Washington, D.C.: 2009.

54. Zegeer, C. V., J. R. Stewart, H. H. Huang, and P. A. Lagerwey. *Safety Effects of Marked vs. Unmarked Crosswalks at Uncontrolled Locations, Executive Summary and Recommended Guidelines.* Report FHWA-RD-01-075. Federal Highway Administration, U.S. Department of Transportation. Washington, D.C.: 2002.

55. FHWA. *MUTCD*, Section 3B.26, FHWA. Manual on Uniform Traffic Control Devices, Federal Highway Administration, Washington, D.C.: 2009.

56. FHWA. *MUTCD*, Section 8C.13, FHWA. Manual on Uniform Traffic Control Devices, Federal Highway Administration, Washington, D.C.: 2009.

57. I.M. Lockwood, "ITE Traffic Calming Definition," *ITE Journal,* Vol. 67, July 1997, pp. 22–24.

58. Ewing, Reid H., *Traffic Calming: State of the Practice*, FHWA-RD-99-135, FHWA and ITE, Washington, D.C.: 1999.

59. FHWA. *MUTCD*, Section 6D, FHWA. Manual on Uniform Traffic Control Devices, Federal Highway Administration, Washington, D.C.: 2009.

60. FHWA, *Separated Bike Lane Planning and Design Guide*, FHWA, Washington, D.C.: 2008.

61. National Association of City Transportation Officials. *Urban Bikeway Design Guide*. Washington, DC, April, 2013, http://nacto.org/wp-content/uploads/2011/03/NACTO_UrbanBikeway_DesignGuide_LRez.pdf

INDEX

CHAPTER 15
BRIDGE

15.0 INTRODUCTION

This chapter addresses the basic issues the roadway designer must consider when designing a roadway project that includes major or minor structures. It also describes the required coordination with the other specialty groups in CDOT.

15.1 SCOPE OF WORK DEVELOPMENT

See the *CDOT Bridge Design Manual* **(1)** Section Policies and Procedures, E, for the minimum project scoping requirements for structures.

The Staff Bridge Branch will play an active and early role in the development of the project-specific activities related to highway structures. The Staff Bridge Unit Leader will designate an experienced Staff Bridge employee to assist the Project Manager in the project scoping. The designated person will normally be the Staff Bridge unit leader whose unit will perform the structural design. When a consultant will do the structural design, the Staff Bridge unit leader assigned to the project will assist in the scoping.

The designated Staff Bridge employee, in conjunction with the Project Manager, will identify the structure-related activities necessary for the project.

The Staff Bridge employee, jointly with the Project Manager, will develop a detailed list of the specific information that is needed by the structural design team from others; (for example: roadway plan and profile, geology report, architectural treatment guidelines) and establish a schedule for receipt of this information by the structural design team.

The Staff Bridge employee and the Project Manager will establish a schedule for the submittals that are to be made by the structural design team. See Section 15.8.

15.2 DEFINITIONS

For additional definitions of structures managed by CDOT and assigned a structure number or structure ID, see the *CDOT Bridge Design Manual* **(1)** Section Policies and Procedures, D.

15.2.1 Major Structures

Major structures are bridges and culverts with a total length greater than 20 ft. measured along the centerline of the roadway between the inside face of abutments, inside faces of the outermost walls of culverts, or spring lines of arches. Major structures also include culverts with multiple pipes where the clear distance between the centerlines of the exterior pipes, plus the radius of each of the exterior pipes, is greater than 20 ft. [*CDOT Bridge Design Manual* **(1)** Section Policies and Procedures, D]

15.2.2 Minor Structures

Minor structures are bridges, culverts, or a group of culverts that have a length greater than or equal to 4 ft. and less than or equal to 20 ft. measured along the centerline of the roadway between the inside face of abutments, inside faces of the outermost walls of culverts, or spring lines of arches. [*CDOT Bridge Design Manual* **(1)** Section Policies and Procedures, D]

15.2.3 Special Inlet or Outlet

Special inlets or outlets are those with features beyond the customary headwalls, wings, and aprons as provided in the *CDOT Standard Plans - M & S Standards* **(2)**. Examples are trash grates, energy dissipaters, trash walls, integral check dams, steeply sloping inverts, varying culvert size at the culvert end, and non-standard apron details.

15.2.4 Standard CBCs Vs. Non-Standard CBCs

Standard concrete box culverts (CBCs) are those covered by the current *CDOT Standard Plans - M & S Standards* **(2)** without modifications. This does not preclude using the *CDOT Standard Plans - M & S Standards* **(2)** as work sheets to provide for custom design and details.

Currently, the *CDOT Standard Plans - M & S Standards* **(2)** are limited to “cell” spans 20 feet and less and opening heights up to 10 feet, typical fill heights between 0 and 20 feet (less for the longer spans and up to 30 feet for triple CBCs), full floor without piles, no change in cross section, and live load of HL-93 Truck, HL-93 Tandem, Colorado Permit Truck, and NRL (national rating load).

Non-standard CBCs are those not described in the *CDOT Standard Plans - M & S Standards* **(2)** in all structural respects. Typically, non-standard CBCs should be used only when standard CBCs cannot reasonably meet the site requirements for loading, span, height, or structural configuration.

15.2.5 Walls

As defined in *CDOT Bridge Design Manual* **(1)** Section Policies and Procedures, D, and *CDOT Retaining and Noise Wall Inspection and Asset Management Manual* **(9)**, walls are classified as follows:

- Retaining Walls: Walls retaining soil measuring at least 4 ft. in height from the finished grade to the top of the wall at any point along the length of the wall.
- Bridge Walls: Retaining walls that contribute to the stability of the bridge or bridge approach. Bridge walls exclude wingwalls and culvert headwalls.
- Noise Walls: Noise walls of all types including other highway partitions and walls that do not typically retain soil.

15.3 ROADWAY ELEMENTS OF DESIGN

15.3.1 Bridge Roadway Width

The curb-to-curb width of a bridge shall carry the full-approach roadway width across the structure. The full-approach roadway width shall include the number and width of travel lanes, width of shoulders, and width of guardrail offset prescribed for the particular functional classification of highway defined for the project. Also, it may include any additional roadway width needed for a median, acceleration or deceleration lanes, other auxiliary lanes, and pavement widening on curves. Where possible, avoid tapering medians, acceleration lanes, deceleration lanes, and other auxiliary lanes across a structure or having the transition for pavement widening on the structure.

Where there is combination curb and gutter construction, the gutter pan width shall be part of the shoulder area on the bridge. The flow-line of roadway curb and gutter and the flow-line of the bridge curb should be aligned. This may be accomplished with a 10-foot transition at the approach to the structure. This policy applies to all structures with either concrete or asphalt approach roadways.

For bridges, other than those on the mainline of an interstate or other divided highway, having approach shoulders less than 8 feet, the guardrail offset shall be 2 feet as specified in M 606-1 of the *CDOT Standard Plans – M & S Standards* **(2)**.

15.3.2 Cross Slope

The cross slope on bridge decks shall be, in all cases, consistent with the cross slope of the adjoining roadway. Where possible avoid having the transition from normal cross slope to full superelevation on a bridge.

15.3.3 Median

The Staff Bridge Branch should be consulted to determine median treatment. Undercrossing roadways may require additional median width to allow placement of bridge pier columns.

15.3.4 Horizontal Alignment

The horizontal alignment of bridges shall be consistent with the adjoining roadway. Where possible, avoid alignments that place spiral curves on structures. More requirements for bridge horizontal alignment can be found in the *CDOT Bridge Design Manual* **(1)** Section 2.2.

15.3.5 Vertical Alignment

The vertical alignment of bridges shall be consistent with the adjoining roadway. In determining vertical alignment, possible structure depths should be discussed with the Project Structural Engineer so that a variety of structure types that are not limited by the lack of sufficient vertical clearance can be considered. Where possible avoid alignments that place the bottoms of sag vertical curves on structures. The recommended minimum grade for drainage is 0.5 percent. For more information refer to the *CDOT Bridge Design Manual* **(1)** Section 2.2.

15.3.6 Bridge Skew Angle

As defined in the *CDOT Bridge Design Manual* **(1)** Section 4.6, bridge skew angles are measured between a line normal to the layout lines (either tangents or chords) or girder lines and the centerlines of bearing of bridge spans or other transverse reference lines. Usually, structures are skewed so that the centerlines of the substructure elements (abutments, piers, culvert walls) are parallel to the feature intersected by the roadway alignment. Where possible avoid horizontal alignments such as spirals that increase the number of different skew angles at the various points of intersection on a bridge. Set the skew angle as close to 0 degrees as possible but less than 50 degrees.

15.3.7 Bridge Sidewalks and Bikeways

The *FHWA Design Guidance and Policy Statement* **(3)** states: "Transportation agencies are encouraged, when possible, to avoid designing walking and bicycling facilities to the minimum standards. For example, shared-use paths that have been designed to minimum width requirements will need retrofits as more people use them. It is more effective to plan for increased usage than to retrofit an older facility. Planning projects for the long-term should anticipate likely future demand for bicycling and walking facilities and not preclude the provision of future improvements."

The clear walkway shall meet current *Public Right of Way Accessibility Guidelines (PROWAG)* **(4)** standards. Additional width (up to 12 feet) may be required in a commercial area or near a school or for a shared pedestrian-bikeway facility. The minimum sidewalk width on a bridge shall be 5 feet.

For high speed, high volume (greater than 45 mph) roadways or those without an approach curb, an approved traffic barrier shall be placed between the travel way and the sidewalk or bikeway.

See Chapters 12 and 14 of this Guide and the *CDOT Bridge Design Manual* **(1)** Section 2.2.5.

15.3.8 Embankment Slopes at Bridge Approaches

Embankment slopes at bridge approaches shall be 2:1 or flatter. For an interstate undercrossing and other high-speed undercrossings, a 4:1 slope shall be placed within the clear zone between the bottom of the outside ditch and the start of the 2:1 slope. Slopes should be designed for adequate drainage (see Section 15.5.2). More detailed information is shown in the *Bridge Design Manual* **(1)** Section 2.2.3 and the *AASHTO Roadside Design Guide* **(5)**.

15.3.9 Clearance to Structures and Obstructions

Minimum horizontal and vertical roadway clearances to structures and obstructions are shown in Table 3-3 of this Guide.

Unusual clearance problems at a structure should be discussed with the Staff Bridge Branch early in the design process so that effective solutions are found. More detailed information on bridge clearances is shown in the *CDOT Bridge Design Manual* **(1)** Section 2.

15.4 ROADWAY DESIGN SUBMITTAL TO BRIDGE/STRUCTURE DESIGNER

15.4.1 Purpose

Normally, the Project Structural Engineer must rely on other members of the design team for site information, horizontal and vertical alignments, hydraulic requirements, and roadway templates. Well-scoped projects and adequate preliminary information eliminate the need for supplemental survey requests. With electronic surveys, special care must be taken during data collection to obtain adequate site information.

15.4.2 Project Scoping

See Section 1 of the *CDOT Project Development Manual* **(6)** and the *CDOT Bridge Design Manual* **(1)** Section Policies and Procedures, E1.

15.4.3 Survey Requests for Bridges

The Project Structural Engineer should be contacted and requested to provide survey requirements prior to making the request for survey. See the *CDOT Survey Manual* **(7)**.

15.4.4 Roadway Design Submittal to Project Structural Engineer

The roadway design submittal should provide sufficient information to locate the structure vertically and horizontally as well as to determine the size of structure required. At a minimum, the following must be provided:

- Typical Sections of Upper and Lower Roadways.
- Roadway Plan and Profile Sheets showing proposed alignments.
- Preliminary Hydraulics Recommendations for any structure over a waterway.
- Bridge Situation Sheet showing topography and contours at 2-foot intervals.
- Locations of all known utilities.
- Any applicable Corridor Design Concepts or special architectural features.
- Preliminary Form 463, Design Data.

Submittals in the electronic format must use the CDOT configuration and include all cross-referenced MicroStation drawings. The files shall not contain LISP files, special character sets, fonts, shape files, or other customized information required to access the drawing files.

Drawings that represent field data surveyed or proposed design alignments should be represented within the drawing files at true scale and in the correct project coordinate locations. The Staff Bridge Branch shall be contacted for other requirements and submittal formats.

All alignments shall be included in the electronic files. Paper copy listings of all points, curves, and horizontal and vertical alignments with stations and coordinates shall be provided. A list of which files contain alignments and surfaces would be helpful.

See the *CDOT Bridge Design Manual* **(1)** Section Policies and Procedures, E1.

15.5 HYDRAULICS REPORTS

15.5.1 Stream and River Crossings

Although hydraulics reports are written by the hydraulics designer, they will be the result of a coordinated, cooperative, multi-disciplinary effort. After a joint site visit by the hydraulics designer and bridge designer, a joint memo will be prepared by the hydraulics designer and sent to the Project Manager stating the concerns, conclusions or issues discussed at the site review. The hydraulics designer will provide the bridge designer the hydraulics information needed to start the design of the structure.

After the borings are taken and analyzed but prior to the submittal of the Foundation Report, the bridge, hydraulics, and geology engineers will discuss bridge site scour conditions.

The Hydraulics Design Engineer will then prepare a Final Hydraulics Report and the Bridge Hydraulic Information Plan Sheets.

For more information, see the *CDOT Drainage Design Manual* **(8)** and the *CDOT Bridge Design Manual* **(1)** Section 2.11 Hydrology and Hydraulics.

15.5.2 Roadside and Bridge Deck Drainage

The Roadway Design Engineer, Hydraulics Design Engineer and Project Structural Engineer should coordinate and analyze roadway and deck drainage requirements. Problems have occurred where drainage was not adequately addressed. Problems ranged from loss of material around guardrail posts to total loss of embankment and slope paving. Consideration of drainage is also needed where the water flows around the abutment wingwalls or ends and where water may cross an expansion device. Discharging drainage from the bridge directly into waterways usually is not permissible and the handling of drainage should be coordinated with the Region Planning/Environmental Program Manager.

15.6 SPECIAL REQUIREMENTS

15.6.1 Permits

Consider the following:

- Coordinate with the Region Planning/Environmental Manager, the materials and geotechnical, and project structural engineers to obtain all necessary environmental permits. See the *CDOT Project Development Manual* **(6)**.
- Construction access to the streambed.
- Right-of-entry permits.
- Temporary easements.
- Maintenance access.

15.6.2 Environmental

It is possible to locate a structure virtually anywhere. However, impact to the environment may weigh in the structure location decision and determine the type of construction. The extent of allowable construction impact to the site must be known to accommodate those limitations in the structure design. Where a structure is located, and what the structure is founded on will affect the construction time required and the cost of a project. Landfills and sites where settlement is likely to occur are less desirable structure locations. Recreational use of the feature spanned by the structure (i.e., kayak, pedestrian, equestrian) as well as the ability of the structure to accommodate recreational use must be thoroughly investigated for integration in the structure design.

Anticipation of dewatering activities for deep foundations should be checked not only for water quality but also the impacts of settlement to surrounding buildings and the roadway.

Whenever environmental concerns need to be accommodated by the structure, they must be made known early, prior to the start of the structure design.

15.6.2.1 Historic Requirements

The activity of determining whether the structure is historic can be a time consuming process. See Section 3.08 of the *CDOT Project Development Manual* **(6)**.

15.6.3 Aesthetics

There is a limit to the amount of aesthetic treatments eligible for Federal funding that can be incorporated into the structure. The designer should consult with the Staff Bridge Branch and FHWA for eligibility. See Section 5.05 of the *CDOT Project Development Manual* **(6)**.

15.6.3.1 Structural Coatings

The appropriate type of coating treatment should be discussed early in the process with Region Maintenance and the Staff Bridge Branch.

15.6.4 Utilities

The location and elevation of all utilities in the vicinity of a structure must be known prior to the start of the final structure design. Frequently, conflicts with utilities can be avoided simply by designing around them. Utilities that must remain in service during construction may require temporary support and additional construction staging. Coordinate with the Staff Bridge Branch if utilities are to be located on the structure and reference the *CDOT Bridge Design Manual* **(1)** Section 2.8.

15.6.5 Construction

15.6.5.1 Detours and Staging

Construction with traffic detoured around the structure site is the most desirable detour method. However, replacement of a structure and structure widening usually requires traffic to be shifted during construction to create a safe work zone. Staging of construction allows a structure to be built in stages while maintaining traffic. Careful planning must go into the construction staging since the feature spanned by the structure will pass beneath throughout construction. Adequate space between the construction stages is required to allow placement of temporary barriers and allow sufficient room to allow work on the structure.

Detour and staging concepts must be developed early because they are needed for the development of the Structure Selection Report and associated bridge general layout.

See Chapter 3 of this Guide and Section 8.02 of the *CDOT Project Development Manual* **(6)**.

15.7 FOUNDATIONS AND STRUCTURES

The Staff Bridge Branch and the Geotechnical Branch will work together to determine the feasible types of the foundation. A foundation investigation request from the Project Structural Engineer will be addressed to the Resident Engineer who will be responsible for site staking and access clearance. Drilling shall not commence until the Geotechnical Engineer has been notified that access is cleared.

See Sections 5.06 and 6.03 of the *CDOT Project Development Manual* **(6)**.

15.8 STRUCTURAL DESIGN SUBMITTALS

The structure design team referred to below is either the Staff Bridge Branch Design Unit or the consultant firm performing the structural design for the project.

The following project submittals will be made by the structure design team. Except for the last bullet, these submittals will be made to the Resident Engineer who will make the necessary distributions. Time frames in parentheses indicate the minimum time required by the project structural engineer to complete the submittal once all necessary information is received. See the *CDOT Bridge Design Manual* **(1)** Section Policies and Procedures, E, for additional information.

If the submittal requires review and comment, the normal time frame allowed for the review is given. Prompt and thorough review of the submittals is necessary to ensure adherence to the project schedule. Changes introduced after the Field Inspection Review can result in a considerable amount of additional design time for the structural design team.

The project structural engineer is responsible for:

- Structure Selection Report. This report is required for all structures and retaining walls. The report shall provide the structure type recommended by the structural design team and shall include structure general layouts (including retaining walls) and preliminary cost estimates. After the report has been reviewed, it shall be revised to include all necessary changes and decisions. This updated report shall identify the final structure type approved for the project (if different than recommended) and provide the associated General Layout and preliminary cost estimate. This report should include pertinent hydraulic and foundation report information. (Two weeks)
- Request for foundation investigation. The structural design team initiates the foundation investigation by identifying the test holes needed as early in the project as is practical. This request shall be sent to the Resident Engineer with a copy to either CDOT's Geotechnical Engineer in the Materials and Geotechnical Branch or the consultant geologist, as applicable.
- General Layouts for the Field Inspection Review. The General Layout in the final Structural Selection Report may be used for the Field Inspection Review set of plans if it has not been revised following review of the Structure Selection Report.
- Advance Plans and Specifications. To reduce or eliminate the need to discuss specific structural design details during the Final Office Review, this optional early review of structural details is made. This review also allows changes that require structural redesign to be made without disrupting the post Final Office Review project schedule. (Three weeks)
- Complete Plans and Specifications for the Final Office Review set of plans.

- Final Plans and Specifications for the advertisement set of plans.
- Submittal of the structural records on the project. This submittal is made to the Bridge Management System unit of the Staff Bridge Branch by the structural design team. This project submittal includes the Structure Selection Report, structural design notes and design check notes, the bridge rating package, and the correspondence file regarding structures.
- Structural Field Packages. Submitted to Resident Engineer for use by the Project Engineer to check quantities and assist with resolving questions about the quantity calculations. This shall include a copy of the foundation report. The field package may be requested at any time after advertisement of the project.

REFERENCES

1. CDOT. *CDOT Bridge Design Manual*, Colorado Department of Transportation, 2018.

2. CDOT. *CDOT Standard Plans - M & S Standards*, Colorado Department of Transportation, 2012.

3. FHWA. *Accommodating Bicycle and Pedestrian Travel: US DOT Policy Statement on Bicycle and Pedestrian Accommodation Regulations and Recommendations*, U.S. Department of Transportation, Federal Highway Administration, Washington, D.C.: 2010. [https://www.fhwa.dot.gov/environment/bicycle_pedestrian/guidance/policy_accom.cfm]

4. ADA. *Public Rights of Way Accessibility Guidelines (PROWAG)*, The Access Board, Washington D.C.: 2011 [https://www.access-board.gov/guidelines-and-standards/streets-sidewalks/public-rights-of-way]

5. AASHTO. *Roadside Design Guide*, American Association of State Highway and Transportation Officials, Washington, D.C.: 2011.

6. CDOT. *CDOT Project Development Manual*, Colorado Department of Transportation, 2013 (with revisions through 2016).

7. CDOT. *CDOT Survey Manual*, Colorado Department of Transportation, 2003 (with revisions through 2016).

8. CDOT. *CDOT Drainage Design Manual*, Colorado Department of Transportation, 2017.

9. CDOT. *CDOT Retaining and Noise Wall Inspection and Asset Management Manual*, Colorado Department of Transportation, 2016

CHAPTER 16
CONSTRUCTION SPECIFICATIONS

16.0 INTRODUCTION

This chapter defines Standard Specifications and Special Provisions. It provides details on the format and guidelines for writing Special Provisions, and describes the approval process for both Standard and Project Special Provisions.

16.1 SPECIFICATIONS - GENERAL

16.1.1 Definition

“Specifications” is a general term applying to all directions, provisions, and requirements pertaining to the performance of the work and payment for the work.

16.1.2 Importance and Characteristics of Well-Written Specifications

Well-written specifications are essential to the efficient construction of a successful project. Well-written specifications inform the Contractor of the work to be performed, the conditions and restrictions on performance of the work, the expected quality of the work, and the manner in which the work will be measured for payment.

With the increased complexity and specialization in modern construction and the need for the Project Engineer to focus on legal requirements and administration, use of the phrase “as directed by the Engineer” should be minimized. Work requirements must be clearly stated in the specifications.

Well-written specifications:

- are clear, concise, and technically correct.
- do not use ambiguous words that could lead to misinterpretation.
- are written using simple words in short, easy to understand sentences.
- use technically correct terms, not slang or “field” words.
- avoid conflicting requirements.
- do not repeat requirements stated elsewhere in the Contract.
- do not explain or provide reasons for a requirement.
- state construction requirements sequentially.
- avoid the use of ambiguous phrases such as “and/or” and “him/her.” Rewriting the sentence can eliminate such phrases.

Furthermore, the phrases “approved by the Engineer” or “accepted by the Engineer” should be avoided. These should be used only when the Engineer will actually accept or approve the work. In such phrases, “approved” and “accepted” are synonymous; there is no difference in the responsibility taken by the Engineer.

16.1.3 Basic Specification Policy

Some of CDOT's established policies for the development and use of construction specifications are described below. These policies are based on Federal and State laws and regulations, CDOT Policy and Procedural Directives, directions from the Chief Engineer, and established CDOT practice.

16.1.3.1 Standards and Specifications Unit

The Standards and Specifications Unit in the CDOT Project Development Branch is charged with overseeing the development and implementation of construction specifications. This unit writes and revises the *CDOT Standard Specifications for Road and Bridge Construction* **(1)** (commonly called the *Standard Specifications*) and *CDOT Supplemental Specifications*, issues Standard Special Provisions, and prepares or reviews Project Special Provisions.

CDOT *Procedural Directive 513.1 - Construction Project Specifications* **(2)**, states that the Standards and Specifications Unit is to review and approve all new Project Special Provisions and newly revised Project Special Provisions that contain significant changes, and initiate a formal review process when necessary. The Standards and Specifications Unit should be given at least two weeks to review proposed Project Special Provisions before they are incorporated into the construction project documents for advertisement.

The CDOT Specification Committee [described in *Procedural Directive 513.1* **(2)**] assists the Standards and Specifications Unit with the review and development of formal specification changes that may be controversial or have a significant impact on the highway construction industry.

16.1.3.2 Liquidated Damages, Penalties, and Incentives

Do not use specifications that assess penalties to the Contractor. The only deductions that can be made from monies due the Contractor are:

- Liquidated damages based on additional engineering costs to the Department.
- Incentives and disincentives based on either the quality of the work or incurred road user costs.
- Price adjustments based on the quality of the work.

In each case, the deduction amount included in the specification must be accurately calculated and documented in the project file. Remediation specified for non-specification work should not be harsh or punitive, but should accurately represent the actual loss of value to the Department or to the road user.

16.1.3.3 Uniformity

CDOT strives to achieve statewide uniformity in the use and application of specifications. Frequent changes to specifications and differences in specifications from project to project and Region to Region lead to misinterpretation, inconsistent enforcement, higher bid prices, and Contractor claims. As much as possible, the *CDOT Standard Specifications for Road and Bridge Construction* **(1)**, Standard Special Provisions and formally issued sample Project Special Provisions and Special Provision Work Sheets should not be changed.

16.1.3.4 Warranties and Guaranties

In accordance with the *Code of Federal Regulations* [23 CFR Part 635.413] **(3)**, warranties or guarantees are allowed on federal aid projects, however, their inclusion within the Contract must be limited to a "specific product or feature" and cannot "place an undue obligation on the Contractor for items or conditions over which the Contractor has no control." Warranties for items of maintenance are not eligible for federal participation and will not be allowed. Allowing the use of a "General Warranty" by making the item non-participating is not an acceptable solution since this is viewed as circumventing the federal requirements. CDOT applies this policy to all projects including those that are not federally funded or are not on the NHS.

Warranties must be for a specific feature or product, and the specification must clearly define the performance indicators and the corrective action required. All warranty specifications shall be sent to the CDOT Standards & Specifications Unit for review and approval four weeks prior to inclusion in a project for advertisement.

16.1.3.5 Proprietary Items

The use of trade or brand names or the direct reference to patented or proprietary materials, specifications, or processes should be avoided in contracts. This applies to all projects, NHS and non-NHS, regardless of funding source. Generic construction specifications should be developed that will obtain the desired results as well as assure competition among equivalent materials or products. There are instances, however, where a particular proprietary product must be specified for use on a project.

If only patented or proprietary products are acceptable, they shall be bid as alternatives with all, or at least a reasonable number of acceptable materials or products listed. A reasonable number would be to specify in the contract two or three equally suitable products and include the term "or approved equal". Note that additional wording shall be added to the specifications to more clearly define the phrase "or approved equal." An example of this is the following:

> For an alternative material to be considered an approved equal, the product shall have been lab tested and field trialed by the National Transportation Product Evaluation Program (NTPEP) and approved by the Department's Materials and Geotechnical Program. Information for the NTPEP evaluation program can be found at http://www.ntpep.org/Pages/RSCPReports.aspx

If a product is on the approved Finding in the Public Interest (FIPI) list it will be noted in the specification and the term "or approved equal" is not to be used. Also, if four or more products are specified, the term "or approved equal" should not be used and neither a FIPI or Certification is necessary. Therefore, when the use of a patented or proprietary (trade name) item is essential and only one item is to be specified in the Contract for synchronization reasons or if no equally suitable alternative exists, then a Certification is necessary. However, if only one item is to be specified in the Contract when there are equally suitable products, or if two or three items without the phrase "or approved equal" are to be specified in the Contract, then a FIPI is necessary.

For more guidance on the use of proprietary items, including development of the certification or FIPI, refer to subsection 2.24.01 of the *Project Development Manual* **(4)**.

16.1.3.6 Materials-Methods Vs. End-Result Specifications

Materials-methods and end-result are the two basic types of construction specifications. Materials-methods specifications describe in detail the materials, workmanship, and processes the Contractor is to use during construction. Materials-methods specifications restrain contractor innovation and obligate the owner to accept the work if the specified materials and processes are used. End-result specifications describe the desired result or quality of the final product to be achieved. End-result specifications encourage contractor innovation and allow the owner to accept or reject the final product. Current CDOT specifications include both types and, in some cases, a combination thereof. End-result specifications are preferred.

Process Control/Owner Acceptance (PC/OA) is a type of end-result specification. PC/OA specifications require the Contractor to perform all testing necessary for control of production while the owner (CDOT) performs the testing necessary to determine acceptance, rejection, or price adjustment of the product. Acceptance, rejection, or price adjustment is usually based on a statistical analysis of the test results. CDOT currently uses PC/OA specifications for pavements.

16.1.3.7 Pay Items

The specifications establish the pay items under which the Department will pay the Contractor for work completed. Readily identifiable and measurable items of work should not be made subsidiary to other items, but should be paid for under separate pay items. Use of lump sum pay items should be minimized. Pay items with subsidiary items and lump sum pay items are difficult for contractors to bid and difficult for the Project Engineer to administer during construction, especially in cases of changed conditions or changed quantities.

Payment for work by force account should be minimized. Force account work involves additional paperwork and often has a higher cost than if the work had been paid for under a bid item.

16.1.3.8 Reference Specifications

AASHTO (American Association of State Highway and Transportation Officials) is the preferred reference for citings. Other national standard references such as ASTM (American Society for Testing and Materials) may be used when there is no AASHTO specification available.

16.1.3.9 Laws, Statutes, and Regulations

Subsection 107.01 of the *CDOT Standard Specifications for Road and Bridge Construction* **(1)** requires the Contractor to be fully informed of, and comply with, all applicable laws and regulations. Generally, specifications that apply, interpret, or enforce laws and regulations should not be used.

16.2 STANDARD SPECIFICATIONS

Work on CDOT construction projects is controlled by the *CDOT Standard Specifications for Road and Bridge Construction* **(1)**. Except where necessary when citing reference specifications (see section 16.1.3.8), the *CDOT Standard Specifications for Road and Bridge Construction* **(1)** contain only English units of measure. See section 16.6.

16.2.1 Organization and Format

The *CDOT Standard Specifications for Road and Bridge Construction* **(1)** are organized into numbered Sections. Sections 101 through 109 contain General Provisions dealing with contracting procedures, general and legal responsibilities of the Contractor, prosecution of the work, control of work and materials, and measurement and payment for the work. Sections 201 through 641 contain construction details, and Sections 701 through 717 contain materials details.

16.2.1.1 Five-Part Format

Each Section of the construction details, Sections 201 through 641, is organized into the following five parts, in the following order:

DESCRIPTION

This part consists of short, succinct statements summarizing the work covered by this Section of the *CDOT Standard Specifications for Road and Bridge Construction* **(1)**. The Description should not contain details, materials or construction requirements, or explanations of measurement and payment.

MATERIALS

This part either specifies the materials requirements the work of this section must meet or refers to subsections in the Materials Details Sections (701 through 717) that contain those requirements.

CONSTRUCTION REQUIREMENTS

This part consists of the required construction procedures or end results of the work to be performed under this Section of the *CDOT Standard Specifications for Road and Bridge Construction* **(1)**. Specific construction details are specified in this part.

METHOD OF MEASUREMENT

This part describes the methods and the units by which the work under this Section of the *CDOT Standard Specifications for Road and Bridge Construction* **(1)** will be measured for payment to the Contractor.

BASIS OF PAYMENT

This part establishes the pay items for work accomplished under this Section of the *CDOT Standard Specifications for Road and Bridge Construction* **(1)** and, when necessary, explains what is included in the payment for those pay items.

16.2.1.2 Subsections

The text of the *CDOT Standard Specifications for Road and Bridge Construction* **(1)** is organized into decimal subsections running consecutively through each Section (the parts are listed in 16.2.1.1). The first subsection is xxx.01, the second xxx.02, etc., where xxx is the Section number.

Subsections are broken into smaller parts ordered by consecutive numerical or alphabetical characters and indented as shown in Figure 16-1.

Numbers in parentheses are also used to identify items in a list, regardless of the placement of the list within the subsection.

See *Figure 16-1, Subsection Organization,* below for an example.

XXX.XX Subsection Name, If Any. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes.

XXX.XY Subsection Name, If Any. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes.

(a) *Subsection name, if any.* This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. THIS IS HOW A LIST IS FORMATTED:

   (1) This is an item in a list.

   (2) This is an item in a list.

   (3) This is an item in a list.

(b) *Subsection name, if any.* This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes.

   1. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes.

   2. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes.

      A. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes.

      B. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes.

         (1) This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes.

         (2) This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes.

(c) *Subsection name, if any.* This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes. This is where the subsection text goes.

Figure 16-1 Subsection Organization

16.3 SUPPLEMENTAL SPECIFICATIONS

Supplemental Specifications are additions and revisions to the *CDOT Standard Specifications for Road and Bridge Construction* **(1)** that are formally adopted subsequent to the issuance of the printed book. Supplemental Specifications apply to all CDOT construction projects in the same manner as the *CDOT Standard Specifications for Road and Bridge Construction* **(1)**. The Contract will clearly identify when Supplemental Specifications are in effect.

16.4 SPECIAL PROVISIONS

Special provisions are additions and revisions to the *Standard* and *Supplemental Specifications* covering conditions unique to an individual project or group of projects. Special provisions apply to a particular construction project only when included in the Contract for that project. Special provisions fall into one of two categories: Standard Special Provisions or Project Special Provisions. Special provisions are developed and implemented according to *Procedural Directive 513.1, Construction Project Specifications* **(2)**.

16.4.1 Organization of Text

The revised or added specification text should be organized under each heading according to the conventions used in the *CDOT Standard Specifications for Road and Bridge Construction* **(1)**.

16.4.2 Margins

The margins used in Special Provisions are 0.75 inch for left and right and 0.5 inch for top and bottom.

16.4.3 Text

Bold and italicized characters should not be used in the body of the text to emphasize or draw attention to a particular requirement. Underlining is not used in the *CDOT Standard Specifications for Road and Bridge Construction* **(1)** and should not be used in Special Provisions.

Titles preceded by (a), (b), etc. should be italicized (see Figure 16-1).

Text should be bold where it would be in the *CDOT Standard Specifications for Road and Bridge Construction* **(1)**. Such locations include section headings, subsection numbers, subsection titles, and table headings.

16.4.4 Standard Special Provisions

Standard Special Provisions are additions and revisions to the Standard and Supplemental Specifications, which are unique to a selected group of projects or are intended for temporary use. Standard Special Provisions are dated and formally issued by the CDOT Project Development Branch with specific instructions for their use. They are to be used without modification. The Standards and Specifications Unit of the Project Development Branch should be contacted if a project has special circumstances that may require modification of a Standard Special Provision.

16.4.4.1 Fonts

The font used for Standard Special Provisions is 10-point Arial.

16.4.5 Project Special Provisions

Project Special Provisions are additions and revisions to the *CDOT Standard Specifications for Road and Bridge Construction* **(1)** and Supplemental Specifications unique to a particular project. The writing style used for Project Special Provisions should be consistent and uniform.

16.4.5.1 Criteria

- Write a Project Special Provision only if the subject has not been adequately covered in the plans, CDOT Standard Specifications for Road and Bridge Construction **(1)**, or Standard Special Provisions.
- Write clear, enforceable requirements that will be interpreted the same way by both the Engineer and the Contractor.
- State the correct pay items. The name of the pay item must be consistent throughout the plans, specifications, and estimate. If the bid item is not listed in the current CDOT Item Book, the Project Manager should contact the Engineering Estimates and Market Analysis Unit and the Standards and Specifications Unit.
- Make sure that Project Special Provisions do not conflict with other parts of the plans and specifications.
- Use end-result rather than materials-methods requirements where possible.
- Specify a requirement; don't make a suggestion or give an explanation.
- Use the verb "will" when stating actions that will be taken by CDOT and "shall" when the action is to be taken by the Contractor. For example, see the following statements in subsection 108.03 of the CDOT Standard Specifications for Road and Bridge Construction **(1)** regarding the project schedule:
  - "The Project Schedule shall show all activities required by all parties to complete the work."[Contractor's responsibility]
  - "The Engineer's review of the schedule will not exceed 10 calendar days. " [Engineer's responsibility]

  See the following statements in subsection 601.05:
  - "Except for Class BZ concrete, the slump of the delivered concrete shall be the slump of the approved concrete mix design plus or minus 2.0 inches." [Contractor's responsibility]
  - "Acceptance will be based solely on the test results of concrete placed on the project." [Engineer's responsibility]
- Use the appropriate Standard Special Provisions as written; don't write a Project Special Provision that covers the same issue without consulting the Standards and Specifications Unit.
- Don't use Project Special Provisions with guaranty or warranty clauses unless they fall within the guidelines described in the *Code of Federal Regulations* 23 CFR Part 635.413 **(3).** Check with the Standards and Specifications Unit to ascertain if policies and procedures have been implemented pertaining to the use of the warranty provision. See section 16.1.3.4.
- Don't use proprietary items except as outlined in section 16.1.3.5.

16.4.5.2 Format and Style

Project Special Provisions should conform to the conventions used in the *CDOT Standard Specifications for Road and Bridge Construction* **(1)**. See the examples for each type described in section 16.4.5.5.

16.4.5.3 Fonts

The font used for Project Special Provisions is 11-point Times New Roman.

16.4.5.4 Titles

The title, capitalized and centered at the top of the page, should identify the section of the *CDOT Standard Specifications for Road and Bridge Construction* **(1)** being revised and the subject of the revision. On multiple page special provisions, the page number pertaining to the special provision should be centered on the first line of the title, on every page. Following is an example:

1

REVISION OF SECTION
105 CONTROL OF WORK

16.4.5.5 Types of Special Provisions

The basic types of special provisions are described below with an example following. Use the example headings, or a variation thereof, for the appropriate type of special provision (xxx represents the section number).

- **Type 1 - Revision of Various Subsections.** Begin a special provision that revises one or more subsections with the following heading:

  Section xxx of the Standard Specifications is hereby revised for this project as follows:

  Follow that statement with the appropriate one or more of the following headings, or variation thereof:

  Subsection xxx.xx shall include the following:

  Delete subsection xxx.xx and replace it with the following:

  In subsection xxx.xx delete the nth paragraph and replace it with the following:

  Subsection xxx.xx, nth paragraph shall include the following:

  In subsection xxx.xx, nth paragraph, delete the nth sentence and replace it with the following:

  When appropriate, follow each heading with the added or revised text.

  Include related changes to separate parts of a section in a single special provision; e.g., when revising CONSTRUCTION REQUIREMENTS and the related MATERIALS part.

Example:

REVISION OF SECTION 202
REMOVAL OF ASPHALT MAT

Section 202 of the Standard Specifications is hereby revised for this project as follows:

Subsection 202.01 shall include the following:

This work includes removal and disposal of existing asphalt mat within the project limits as shown on the plans or at locations directed by the Engineer.

In subsection 202.02 delete the seventh paragraph and replace with the following:

The existing asphalt mat which varies in thickness from 2.5 inches to 6 inches shall be removed in a manner that minimizes contamination of the removed mat with underlying material. The removed mat shall become the property of the Contractor and shall be either disposed of outside the project site, or used in one or more of the following ways:

(1) Used in embankment construction in accordance with subsection 203.06.
(2) Placed in bottom of fills as approved by the Engineer.
(3) Recycled into the hot mix asphalt.
(4) Placed in the subgrade soft spots as directed by the Engineer.

Subsection 202.11 shall include the following:

The removal of the existing asphalt mat will be measured by the square yard of mat removed to the required depth and accepted.

Subsection 202.12 shall include the following:

Payment will be made under:

Pay Item	**Pay Unit**
Removal of Asphalt Mat	Square Yard

Unless otherwise specified in the Contract, the disposal of the asphalt mat or its use in other locations on the project will not be measured and paid for separately, but shall be included in the work.

- **Type 2 - Deletion and Replacement of an Entire Section.** Begin a special provision that deletes and replaces an entire Section with the following:

  Section xxx of the Standard Specifications is hereby deleted for this project and replaced with the following:

Follow this statement with the revised text of the Section. Organize the text into the five main parts: DESCRIPTION, MATERIALS, CONSTRUCTION REQUIREMENTS, METHOD OF MEASUREMENT, and BASIS OF PAYMENT.

Example:

REVISION OF SECTION
306 RECONDITIONING

Section 306 of the Standard Specifications is hereby deleted for this project and replaced with the following:

DESCRIPTION

306.01 This work consists of ripping and pulverizing the existing asphalt mat, regrading and compacting the subgrade with moisture and density control, and placing the pulverized bituminous material as a modified base course atop the subgrade, in accordance with the specifications, at locations shown, and in conformity with the details shown on the plans or as staked.

CONSTRUCTION REQUIREMENTS

306.02 The existing mat shall be ripped, pulverized, and placed in windrows. The maximum particle size of the pulverized bituminous material shall be 1.5 inches.

The top 4.5 inches of the subgrade material shall then be removed and disposed of at the location designated on the plans. The top 6 inches of the remaining subgrade material shall be scarified, shaped, and compacted using moisture and density control. The subgrade surface shall not vary above or below the lines and grades staked by more than 1 inch. The surface will be tested prior to placement of the pulverized bituminous material.

The pulverized bituminous material shall then be placed as shown on the plans and compacted using moisture and density control.

METHOD OF MEASUREMENT

306.03 Reconditioning will be measured by the square yard of roadway treated, complete and accepted.

BASIS OF PAYMENT

306.04 The accepted quantities of reconditioning will be paid for at the contract unit price per square yard for reconditioning.

Payment will be made under:

Pay Item	**Pay Unit**
Reconditioning	Square Yard

Payment for reconditioning will be full compensation for all work necessary to complete the item including ripping and pulverizing the existing asphalt mat, excavation and disposal of subgrade material, scarifying and compacting the subgrade, placing and compacting the pulverized bituminous material, blading, shaping, haul, and water.

- **Type 3 - Addition of a New Section.** Begin a special provision that adds a new specification Section with the following:

  Section xxx is hereby added to the Standard Specifications for this project as follows:

  Follow this statement with the text of the new Section. Organize the text into the five main parts: DESCRIPTION, MATERIALS, CONSTRUCTION REQUIREMENTS, METHOD OF MEASUREMENT, and BASIS OF PAYMENT.

  Example:

SECTION 621
TEMPORARY BRIDGE

Section 621 is hereby added to the Standard Specifications for this project as follows:

DESCRIPTION

621.01 This item includes loading, transporting, erecting, maintaining, removing, and returning the temporary bridge.

MATERIALS

621.02 The temporary bridge is a Bailey Bridge in the possession of the Department. The Department will not charge a rental fee for the use of this bridge on this project.

CONSTRUCTION REQUIREMENTS

621.03 The Contractor shall load and return the temporary bridge at the following site:

[INSERT SPECIFIC LOCATION]

The Contractor shall make arrangements with Department maintenance personnel at least five days prior to the loading and returning dates.

The temporary bridge shall be erected at the location and in conformity with the lines and grades shown on the plans or established.

The Contractor shall replace all structural parts that are missing or damaged when the bridge is returned. The Contractor shall band all components together before returning the bridge for storage.

The Contractor shall return the temporary bridge within 30 days after the new structure is opened to traffic.

METHOD OF MEASUREMENT

621.04 Temporary Bridge will not be measured, but will be paid for on a lump sum basis.

BASIS OF PAYMENT

621.05 The completed and accepted work for the temporary bridge will be paid for at the contract lump sum price. This price shall include all labor, equipment, and materials required to load, transport, erect, maintain, remove, and return the temporary bridge.

Payment will be made under:

Pay Item	**Pay Unit**
Temporary Bridge	Lump Sum

- **Type 4 - Addition of Changes Not Tied to Specific Subsections.** Begin a special provision that adds text throughout the Section and that does not tie in well to the existing subsections (such as requirements for a new item or type of construction) with the following:

  Section xxx is hereby revised for this project to include the following:

  Follow this statement with the new text organized into the five main parts: DESCRIPTION, MATERIALS, CONSTRUCTION REQUIREMENTS, METHOD OF MEASUREMENT, and BASIS OF PAYMENT.

  Special provision Type 1 is the preferred and most commonly used type of special provision. Samples of each type of special provision appear in subsection 16.8.

  Example:

REVISION OF SECTION 210
RESET IMPACT ATTENUATOR

Section 210 of the Standard Specifications is hereby revised for this project to include the following:

DESCRIPTION

This work consists of resetting impact attenuators in accordance with these specifications and in conformity with the lines and details shown on the plans or established.

MATERIALS

The impact attenuators are the types shown at the various locations on the plans.

CONSTRUCTION REQUIREMENTS

The site shall be prepared to receive the reset impact attenuator by filling, excavating, smoothing and all other work necessary for the proper installation of the attenuator.

The impact attenuator shall be installed in accordance with the manufacturer's recommendations.

METHOD OF MEASUREMENT

Reset impact attenuator will be measured by the number of attenuators as shown on plans, reset and accepted, including site preparation and all necessary hardware.

BASIS OF PAYMENT

The accepted quantities will be paid for at the contract unit price for the pay item listed below.

Payment will be made under:

Pay Item	**Pay Unit**
Reset Impact Attenuator	Each

These are the most common types of special provisions; there are variations.

16.4.5.6 Revised or Added Specification Text

Special provisions revising any of the Sections 201 through 641 should be written so that the revised or added specification text is incorporated into the appropriate subsections under one or more of the five main parts. New main parts should not be established except in the rare instance of adding an item for design to be performed by the Contractor.

The organization used for the *CDOT Standard Specifications for Road and Bridge Construction* **(1)** should be followed for the added or revised text of Project Special Provisions. The part of the subsection being revised should be identified and the new or revised text should be made to fit that part. The text or breakdown character should start at the left margin and not be indented.

16.4.6 Use of New or Revised Project Special Provisions

New and newly revised Project Special Provisions that contain significant changes must be reviewed by the Standards and Specifications Unit in the Project Development Branch. These should be submitted electronically in the format described above and with sufficient review time (normally two weeks). The Project Manager should be prepared to explain the engineering or project management considerations that justify the use of the Project Special Provision.

The Standards and Specifications Unit will review the proposed special provision for conformance to CDOT policy and FHWA regulations, potential controversy, clarity, grammar, punctuation, and format. The Standards and Specifications Unit will respond with approval, suggested changes, or a statement that the special provision should not be used. When the Standards and Specifications Unit determines that a proposed special provision is controversial or addresses an issue with broad impact, it may initiate a more formal review process to be completed before the proposed special provision can be used on CDOT construction projects.

If the proposed Project Special Provision covers an issue that could have statewide implications, the Branch Manager or Region Transportation Director should request review by the appropriate CDOT Technical Committee or submit a Form 1215 - *Submittal of New Specification or Specification Change* **(8)** to the Standards and Specifications Unit.

16.4.7 Special Provision Package

The Contract documents for each CDOT construction project include a set of special provisions accompanying the plan sheets. This set of special provisions consists of an index of the Project Special Provisions and an index of the applicable Standard Special Provisions followed by the Project and Standard Special Provisions. The project manager inserts the Project Special Provisions listed on the index. CDOT has created a tool for putting the special provision package together. The tool and its instructions can be found on the CDOT website at: https://www.codot.gov/business/designsupport/cdot-construction-specifications/2017-construction-standard-specs/psp_specs_writing_program .

When preparing the special provision package for a project, the project number and code should be listed on the left and the date on the right at the top of each Project Special Provision page.

The page number should be centered at the bottom of each page. The Index and Project Special Provision pages should be numbered consecutively, beginning with Page 1.

16.5 CONSTRUCTION SPECIFICATIONS WEBSITE

The Standards and Specifications Unit maintains Special Provisions on the CDOT website.

16.5.1 Accessing the Website

The CDOT website address is https://www.codot.gov. The Specifications page on the CDOT website is located at https://www.codot.gov/business/designsupport/cdot-construction-specifications .

16.5.2 Contents of the Website

The Specifications page on the CDOT website contains:

- Standard Specifications Text
- Current Standard Special Provisions
- Project Special Provision Work Sheets
- Sample Project Special Provisions
- Materials Specifications Check List
- Design/Build Special Provisions
- Fuel Cost Adjustment
- Asphalt Cement Cost Adjustment
- Past Davis-Bacon Minimum Wage Decisions
- Innovative Contract Provisions
- Phased Funding Special Provisions
- Warranted HBP Special Provisions
- Significant Changes found in the *CDOT Standard Specifications for Road and Bridge Construction* **(1)**

The following information is also available:

- Creating a Special Provision Package for a CDOT Project
- Guidelines for Writing Construction Specifications (this document)
- Specification Changes Under Consideration

16.5.2.1 Project Special Provision Work Sheets

Work sheets available on the website include those for frequently used Project Special Provisions and instructions for index pages, Notice to Bidders, Commencement and Completion of Work, and Traffic Control Plan - General.

16.5.3 Updates

The Standards and Specifications Unit notifies users of updates to the website by e-mail.

16.6 USE OF METRIC AND ENGLISH UNITS

The *CDOT Standard Specifications for Road and Bridge Construction* **(1)** and Standard Special Provisions used with it contain only English units. Project Special Provisions should contain only English units, except where metric units are required to conform to reference specifications.

16.7 WRITING STYLE

Traditionally, specifications are written in the indicative mood, either active or passive voice.

- Active voice:

  The Contractor shall place the aggregate to a depth of 6 inches and compact it to a density of 95 percent.

- Passive voice:

  The aggregate shall be placed to a depth of 6 inches and compacted to a density of 95 percent.

Some states have rewritten their standard specifications in the imperative mood, active voice. This style of writing replaces the lengthy "the Contractor shall" sentences with short sentences giving direct instructions.

- Imperative mood, active voice:

  Place the aggregate to a depth of 6 inches and compact it to a density of 95 percent.

However, CDOT has not adopted the imperative mood style in the *CDOT Standard Specifications for Road and Bridge Construction* **(1)**. The book is written in the indicative mood, either active voice (where possible) or passive voice (where necessary).

Special provisions should match the style of the *CDOT Standard Specifications for Road and Bridge Construction* **(1)**. In special provisions, use short simple sentences in the active voice wherever possible. Use the imperative mood only if it is preceded by an introductory statement clarifying that the text makes a requirement on the Contractor. An example that appears in subsection 209.05 of the *CDOT Standard Specifications for Road and Bridge Construction* **(1)** is the following:

> Magnesium Chloride dust palliative shall be applied as follows:
>
> (1) Scarify the top 2 inches of the existing road surface and wet with water to approximately four percent moisture content, or as directed.
> (2) Apply the magnesium chloride dust palliative in two applications of 0.25 gallon per square yard in each application.
> (3) Allow to soak for 30 minutes after each application.
> (4) Roll the surface with a pneumatic tire roller, as specified in the Contract.
> (5) Do not permit traffic on the treated surface until approved.

Other protocols for grammar, syntax, and format that have been applied in the Standard Specifications and that should be applied to special provisions appear in Table 16-1.

ITEM	IN TEXT	IN TABLES (and tabular lists)	IN LISTS (consisting of text)
Numbers	For counts from 1 to10 use words: three hours, four posts; Counts over 10 use digits: 24 hours, 14 posts; where one of each is related, use digits for both: 6 to 12 hours. For dimensions & measurements use digits: 6 inches, 7 cubic yards.	Use digits	For counts from 1 to10 use words: three hours, four posts; Counts over 10 use digits: 24 hours, 14 posts; where one of each is related, use digits for both: 6 to 12 hours. For dimensions & measurements use digits: 6 inches, 7 cubic yards.
Ordinal Numbers	Use words: first, fifth, twentieth	Use symbols: 1^{st}, 5^{th}, 20^{th}	Use words: first, fifth, twentieth
Large numbers & money	Do not reiterate in Parentheses: $80,000 – not $80,000 (eighty thousand dollars)	Do not reiterate in Parentheses: $80,000 – not $80,000 (eighty thousand dollars)	Do not reiterate in Parentheses: $80,000 – not $80,000 (eighty thousand dollars)
Dimensions	Use words: foot, yard, inches	May use abbreviation (ft., yd.) or symbol (', ")	Use words
Areas	Use words: square foot, square yard	May use abbreviation: sq. ft., sq. yd.	Use words
Volumes	Use words: cubic yard, cubic feet, gallons	May use abbreviation: cu. yds. cu. ft., gal.	Use words
Densities/ rates	Use words: pounds per cubic yard, gallons per square yard	May use abbreviations: lbs./cu. yd., gal./sq. yd.	Use words
Temperature	Use symbol: °F	Use symbol: °F	Use symbol: °F
Ranges	Use "to" not "-": 180 to 190 °F, 6 to 12 inches	Use "to" or "-": 180 - 190 °F, 6 to 12"	Use "to" not "-": 180 to 190 °F, 6 to 12 inches
SI sieve sizes	Use symbols: 19.0 mm, 300 µm	Use symbols: 19.0 mm, 300 µm	Use symbols: 19.0 mm, 300 µm
SAE sieve sizes	Use words: 2 inch, ½ inch, No. 30, No. 100	Use symbols: 2", ½", #30, #100	Use symbols: 2", ½", #30, #100
Right of Way	Use words or abbreviation: Right of Way, ROW	Use abbreviation: ROW	Use abbreviation: ROW
Dual Units, e.g. sieve sizes	SI first with SAE in parentheses: 25.0 mm (1 inch)	SI first with SAE in parentheses: 25.0 mm (1 inch)	SI first with SAE in parentheses: 25.0 mm (1 inch)
CDOT Forms	Use just the form No.: Form 463	Use just the form No.: Form 463	Use just the form No.: Form 463
Other Forms	Identify the originating organization: FHWA Form 1273	Identify the originating organization: FHWA Form 1273	Identify the originating organization: FHWA Form 1273
Percentages	Use word: 12 percent, 25 percent	Use symbol: 12 %, 25%	Use word: 12 percent, 25 percent
Ratios	Use colon: 1:1, 1½:1	Use colon: 1:1, 1½:1	Use colon: 1:1, 1½:1

Table 16-1 Spec Book Grammar, Syntax, and Format Protocol

ITEM	IN TEXT	IN TABLES (and tabular lists)	IN LISTS (consisting of text)
Use of "and/or"	Do not use "and/or": alternatives are: "a, b, or both" and "a, b, c, or a combination thereof". Sometimes "a, b, or c" works just as well.	Does not usually appear in tables.	Do not use "and/or": alternatives are: "a, b, or both" and "a, b, c, or a combination thereof". Sometimes "a, b, or c" works just as well.
Use of "noun(s)"	Avoid use of "noun(s)". Can often use just the singular or plural; or rewrite the sentence: workers [however many], each worker.	Does not usually appear in tables.	Avoid use of "noun(s)". Can often use just the singular or plural; or rewrite the sentence: workers [however many], each worker.
Use of "under Item XXX"	Avoid use of this construction. Instead of "will be paid for under Item 613" use "will be paid for in accordance with Section 613."	Does not usually appear in tables.	Avoid use of this construction. Instead of "will be paid for under Item 613" use "will be paid for in accordance with Section 613."
Use of the word "any".	Avoid using the word "any", especially where it can be inferred that the Contractor chooses an alternative. Any is a vague word that can mean: all, a selected alternative, every, a specific one, etc. Examples: "The Contractor shall remove any laitance" reads better as "The Contractor shall remove all laitance"; "Any source of borrow other than an available source will be known as a contractor source" reads better as " Sources of borrow other than an available source will be known as Contractor sources."	Does not usually appear in tables.	Avoid using the word "any", especially where it can be inferred that the Contractor chooses an alternative. Any is a vague word that can mean: all, a selected alternative, every, a specific one, etc. Examples: "The Contractor shall remove any laitance" reads better as "The Contractor shall remove all laitance"; "Any source of borrow other than an available source will be known as a contractor source" reads better as " Sources of borrow other than an available source will be known as Contractor sources."
Use of shall & will	When the helping verb "shall" is used in a sentence, it normally indicates that the Contractor is required to perform the stated action in the manner prescribed. When the helping verb "will" is used in a sentence, it normally indicates an action that is intended to be performed by the Engineer or his representative as appropriate.	Does not usually appear in tables.	When the helping verb "shall" is used in a sentence, it normally indicates that the Contractor is required to perform the stated action in the manner prescribed. When the helping verb "will" is used in a sentence, it normally indicates an action that is intended to be performed by the Engineer or his representative as appropriate.
Referring to plans and specifications	In most cases, refer to "in the Contract". When it is necessary to refer specifically to plans or specifications use: "in the specifications" or "on the plans".	Does not usually appear in tables.	In most cases, refer to "in the Contract". When it is necessary to refer specifically to plans or specifications use: "in the specifications" or "on the plans".

Table 16-1 Spec Book Grammar, Syntax, and Format Protocol (Continued)

REFERENCES

1. CDOT. *Standard Specifications for Road and Bridge Construction*, Colorado State Department of Transportation.

2. CDOT. Procedural Directive 513.1, *Construction Project Specifications*, Colorado State Department of Transportation, 2016. [https://www.codot.gov/business/designsupport/cdot-construction-specifications/2017-construction-standard-specs/specs-changes-under-consideration/pd-513-1-review/view]

3. FHWA. *Code of Federal Regulations*, Title 23, Part 635, 2018. [https://www.fhwa.dot.gov/legsregs/directives/cfr23toc.htm]

4. CDOT. *Project Development Manual*. [https://www.codot.gov/business/designsupport/bulletins_manuals/project-development-manual/revs-to-project-manual]

5. USC. *United States Code*, Title 23, Chapter 1, Section 112. [https://www.gpo.gov/fdsys/browse/collectionUScode.action?collectionCode=USCODE]

6. CDOT. *Innovative Contracting (Design-Build and CM/GC)*, Colorado State Department of Transportation. [https://www.codot.gov/business/designsupport/innovative-contracting-and-design-build]

7. CDOT. *Innovative Contract Provisions*, Colorado State Department of Transportation. [https://www.codot.gov/business/designsupport/cdot-construction-specifications/2017-construction-standard-specs/innovative-specs]

8. CDOT. Form 1215 - *Submittal of New Specification or Specification Change*. [https://www.codot.gov/library/forms/cdot1215.pdf]

CHAPTER 17

This chapter is currently under development.

CHAPTER 18
NOISE GUIDE

18.0 INTRODUCTION

The problem of noise generated by highway traffic involves physical, physiological, and psychological factors that cause varying reactions by the public. Highway traffic noise should be considered in the location and design of roadways.

This chapter is intended to help designers identify issues related to highway traffic noise, understand the applicable federal and state regulations and guidelines, analyze traffic noise for specific projects, and select and implement noise mitigation measures.

From a project's inception, noise mitigation measures may need to be considered as a part of the project. As early in the process as possible, consult the appropriate environmental documentation, if available, to determine if mitigation commitments were made. Contact the Region Environmental Staff for assistance with these determinations and the appropriate regulatory requirements.

This chapter contains a summary of basic concepts and supplements existing published material. If more detail is needed concerning any of the following specific subjects, consult the references provided at the end of this chapter. If existing regulation or guidance is revised, or if additional regulation or guidance is published after the date this chapter was published, the new material takes precedence.

18.1 NOISE FUNDAMENTALS

Noise is defined as unwanted or excessive sound. Sound (or noise) levels are measured in units of decibels (dB), which are measured on a logarithmic scale which condenses a large range of several magnitudes of sound pressure levels. For the purposes of highway traffic, an “A-scale” weighting is applied to noise levels because the human ear does not perceive all sound frequencies equally. These are referred to as A-weighted decibels (dBA).

Because the sound intensity of highway traffic is not constant, a descriptor is needed to describe the source in a steady-state condition. The most common descriptor, which is used for CDOT projects, is the hourly equivalent sound level ($L_{eq}(h)$). For highway traffic noise analyses, noise levels are expressed in terms of hourly equivalent A-weighted decibels and expressed in this manner: dBA $L_{eq}(h)$.

Since noise levels in the decibel scale are logarithmic, they cannot be added arithmetically. For example, adding two 70-decibel sources results in a noise level of 73 decibels. Any doubling of a noise source, such as doubling the volume of traffic on a roadway or moving the existing traffic twice as close to a neighborhood, increases the overall decibel level 3 decibels. Studies have shown that a 3-decibel change in noise levels is barely detectable by the human ear, even though the overall sound energy has doubled. It normally takes a 5-decibel change in noise levels to be perceptible to most people. A 10-decibel change in noise levels is normally perceived as either a doubling or a halving of the perceived "loudness" of noise levels. Frequency changes, however, may be detectable by people even if the overall decibel levels are unchanged.

For highway projects, the noise level of interest is the worst-hour noise equivalent level. This is the time of day when noise levels are highest. It is used for comparison with impact criteria to determine noise impacts. It is also when the highest number of vehicles is traveling at the highest possible speed. This is not necessarily the peak travel hour or rush hour, because there may be periods of congestion when traffic tends to slow, resulting in lower noise levels. For highways that tend to be congested at peak hour, the worst-noise hour is the period either just before or just after peak hour.

18.2 NOISE REGULATIONS AND ANALYSIS REQUIREMENTS

The regulations that govern highway traffic noise for Federal-aid projects are contained in Part 772 of Title 23 of the Code of Federal Regulations (23 CFR 772) **(1)**. Regulation 23 CFR 772 describes methods that must be followed in the evaluation and mitigation of highway traffic noise in Federal-aid highway projects. The FHWA will not approve plans or specifications for any federally aided highway project unless the project includes noise abatement measures, if the measures have been deemed to be feasible and reasonable to adequately reduce noise impacts. When warranted, noise mitigation is to be considered as an integral component of the total project development process and incorporated as such.

Projects that fall under 23 CFR 772 are classified as Type I, Type II or Type III.

The project type will be determined in conjunction with a CDOT noise specialist. Type I definitions are in 23 CFR 772 and *CDOT Noise Analysis and Abatement Guidelines* (2). Projects that are or may be Type I include but are not limited to:

- Constructing a new road on a new location
- Adding a lane, auxiliary lane, or ramp
- Changing the elevation of the road or ramp
- Moving the road closer to receptors
- Adding a new or substantial alteration of a weigh station, rest stop, ride-share lot, or toll plaza

Type II projects are defined as the construction of noise abatement on existing highways ("retrofit" projects) in absence of major highway construction. State funding has been unavailable for this program since 1999.

A Type III project is any project that does not meet criteria for either a Type I or Type II project. Noise analyses are not required for these non-noise sensitive projects. Examples of Type III projects include resurfacing, bridge rehabilitation including bridge reconstruction with minor non-capacity widening, and most shoulder and maintenance projects.

A noise analysis study is required for all Type I projects if noise sensitive properties (receptors) are present in the project study zone, which is defined as a 500-foot halo from the proposed edge of the traveled lane(s). A noise sensitive receptor is any location for which highway traffic noise may be detrimental to the outdoor enjoyment and functional use of the property. Noise sensitive receptors include residences, parks, hotels, schools, and noise-sensitive businesses such as recording studios. A complete list of receptor types is in Table 1 of 23 CFR 772 and Table 18-1 of this document.

The purpose of a noise analysis is to identify receptors that are impacted by noise. Noise is determined at existing conditions and is projected for the project design year, which is usually 20 years in the future. As defined in the regulations, a traffic noise impact occurs when projected noise levels approach or exceed the Noise Abatement Criteria (NAC) or when the noise from projected traffic levels substantially exceeds the existing noise levels. NACs are noise thresholds in units of dBA and vary by land use category. A substantial increase in noise level is defined as being 10 dBA or more.

An impact due to a substantial increase in noise is rarely encountered in situations other than the construction of a new highway on a new location. Table 18-1 identifies the NAC levels for different land use categories.

Table 18-1: CDOT Noise Abatement Criteria for Receptor Activity Descriptions

Activity Category	Activity Leq(h)[2]	Evaluation Location	Activity Description
A	56	Exterior	Lands on which serenity and quiet are of extraordinary significance and serve an important public need and where the preservation of those qualities is essential if the area is to continue to serve its intended purpose.
B[1]	66	Exterior	Residential
C[1]	66	Exterior	Active sport areas, amphitheaters, auditoriums, campgrounds, cemeteries, day care centers, hospitals, libraries, medical facilities, parks, picnic areas, places of worship, playgrounds, public meeting rooms, public or nonprofit institutional structures, radio studios, recording studios, recreational areas, Section 4(f) sites, schools, television studios, trails, and trail crossings.
D	51	Interior	Auditoriums, day care centers, hospitals, libraries, medical facilities, places of worship, public meeting rooms, public or nonprofit institutional structures, radio studios, recording studios, schools, and television studios.
E[1]	71	Exterior	Hotels, motels, time-share resorts, vacation rental properties, offices, restaurants/bars, and other developed lands, properties or activities not included in A-D or F.
F	NA	NA	Agriculture, airports, bus yards, emergency services, industrial, logging, maintenance facilities, manufacturing, mining, rail yards, retail facilities, ship yards, utilities (water resources, water treatment, electrical), and warehousing.
G	NA	NA	Undeveloped lands that are not permitted for development.

1 Includes undeveloped lands permitted for this activity category.
2 Hourly A-weighted sound level in dBA; reflects values that are 1 dBA less than 23 CFR 772 values

The NAC values in Table 18-1 are based on speech communication interference. These noise levels are not legal standards or mitigation design goals, but are the levels for which mitigation must be considered. The levels shown reflect the CDOT approach criterion, which are 1 dBA less than FHWA levels. The most common land use activity category included in noise analyses is Category B, residences, which has a NAC of 66 dBA.

Noise regulations require that any receptor determined to be impacted by noise, based on modeling, is entitled to mitigation consideration. Mitigation must be provided if it is found to be feasible and reasonable.

The feasibility determination considers the physical constructability of the abatement measure. Factors include construction complexity, required area needed for foundations or equipment, and compatibility with utilities and drainage. Complex construction techniques require more time to install, more cost in labor and equipment, and may cost more to maintain. For a mitigation measure to be feasible, it must be able to be constructible to normal engineering standards to provide a perceivable noise reduction of at least 5 dBA at a minimum of one receptor. Walls cannot be more than 20 feet in height. Walls must not cause unsafe visibility or maintenance concerns such as obscuring egress visibility or creating a shadow zone resulting in persistent icing within a travel lane.

Any mitigation that is deemed to be feasible must also meet the three reasonableness socio-economic criteria before it is implemented: a minimum noise reduction, cost-effectiveness, and the preference of the benefited residents and property owners to have noise abatement measures constructed. The minimum noise reduction is a noise reduction design goal of at least 7 dBA at a minimum of one receptor. Documentation of the analysis and mitigation decision-making process must be clear and complete.

18.3 CDOT NOISE ANALYSIS AND ABATEMENT GUIDELINES

Applicable procedures for conducting a project level highway traffic noise analysis are detailed in the *CDOT Noise Analysis and Abatement Guidelines*. The guidelines provide a consistent and equitable approach and decision-making process for addressing highway traffic noise on highway projects. These guidelines are compliant with 23 CFR 772 and were approved by FHWA. Additional guidance is available from FHWA **(3 through 10)** and CDOT *Procedural Directive 1601: Interchange Approval Process*. **(11)**

The *CDOT Noise Analysis and Abatement Guidelines* include a sample Benefited Receptor Preference Survey and CDOT Form 1209, the Noise Abatement Determination worksheet. The survey is the third required criteria of the reasonableness test. Form 1209 documents the feasibility and reasonableness determination for each evaluated mitigation measure.

The guidelines also discuss public involvement, coordination with local officials, construction considerations, and National Environmental Policy Act (NEPA) documentation requirements. NEPA requirements are also discussed in the CDOT Environmental Stewardship Guide. **(12)**

Information provided in this chapter gives a general overview of noise regulations and analysis procedures. However, the *CDOT Noise Analysis and Abatement Guidelines* describe these issues in much greater detail and should be consulted, especially if there are any issues regarding a specific project.

18.4 HIGHWAY TRAFFIC NOISE MITIGATION MEASURES

FHWA allows use of several noise mitigation measures, but only requires that noise barriers (berms, walls, or a combination) be considered as mitigation for impacted receptors. If analyzing another mitigation measure, a determination as to the validity and practicality of successfully implementing the measures must be made. CDOT guidelines are developed to be in compliance with the Federal policies and regulations. CDOT guidelines are approved by the FHWA as the method of determining and abating noise in Colorado on Federal-aid projects.

Vegetation does not function as noise mitigation unless it consists of 200 to 300 feet of dense, permanent foliage ground floor to treetop coverage of at least 16 feet high. While vegetation can be of aesthetic and psychological benefit, and can enhance an area where it is placed and successfully maintained, it is usually only provided for visual, privacy, or aesthetic treatment.

The rest of this section provides information about mitigation measures that are allowed but not required by FHWA.

18.4.1 Traffic Management Measures

Traffic management measures may reduce traffic noise levels. Examples include:

- Lane use restrictions for certain vehicle types
- Time use restrictions for certain vehicle types
- A combination of lane and time use restrictions

- Installation and proper timing of traffic control devices
- Reduction of speed limits.

The feasibility of providing specified truck routes or utilizing lane restrictions on truck usage should be determined on a case-by-case basis.

Lowering speed limits can reduce noise and is cited by the public as a mitigation method. However, generally a speed reduction of at least 20 mph is needed to sufficiently decrease noise levels. Therefore, this option has operational issues.

18.4.2 Alteration of Horizontal or Vertical Alignments

Altering the design of the roadway can be very effective in reducing noise levels and noise impacts. Although several techniques are possible, certain projects and areas will not be conducive to some or any of these mitigation measures. In most cases, reductions in noise levels are based on increasing the distance between the roadway and the receptors, or by providing for terrain between the highway and receptors.

Proper siting of highway alignment in relationship to noise sensitive areas is the most effective way to reduce noise impacts. Any increase in the distance between the highway and receptors will reduce noise levels. For divided highways, use of natural terrain features and barriers to separate the individual roadway sections can provide additional noise reduction.

If the roadway can be depressed through a cut section, noise levels will be reduced for the area that is shielded by the adjacent slope. This is most effective when vehicles can be screened from view. Elevated sections of roadway create a shadow zone for receptors that are close to the embankment or structure. Noise is reduced in shadow zones. However, elevated sections may cause slight noise increases to receptors farther from the roadway due to the loss of shielding by adjacent structures.

In some cases, especially where there is a high percentage of heavy truck traffic, grade reductions can reduce noise levels due to the reduction in the need for vehicles (especially heavy trucks) to accelerate and decelerate. This is particularly useful on long downgrades where trucks are inclined to engage their engine compression (e.g., “Jake”) brakes. However, there is a tradeoff with this option: gentler grades have the potential to increase noise levels due to the longer exposure time.

Pavement type is often cited as a possible means to reduce highway traffic noise. The majority of noise emitted from highways is due to the tire-pavement interaction. Research on this issue has been ongoing since the 1970s. The effect of different pavements over long periods, 20 years or more, has still not been clearly established. Studies have indicated that open-graded asphalt pavements, when first placed, can produce a benefit of 2 to 5 dBA of noise level reduction. However, after 6 months to 2 years, the aggregate becomes polished and voids in the pavement fill so noise reduction benefits are lost. Concrete pavement, while perhaps louder than asphalt when it is initially placed, will become quieter over time. Longitudinal tining or diamond grinding of the concrete, where possible, results in reduced noise levels compared to smooth concrete surfaces. Transverse tining, or tining of the concrete perpendicular to the direction of travel, creates an annoying high-pitched whine and should not be used.

FHWA policy says that pavement type cannot be used as noise mitigation in lieu of other feasible and reasonable noise abatement measures. Noise mitigation must provide a “readily perceptible” noise reduction over a long period of time (20 years), and it is difficult to forecast the overall pavement condition under a future condition. Noise may be used as a factor to be considered in pavement selection if the life cycle cost analysis among pavement options yields similar results.

18.4.3 Acquisition of Property or Property Rights

In undeveloped areas, the acquisition of additional right of way or development rights can be an effective means of providing a buffer between the highway and any future land development. The purpose of this practice is to prevent dwellings from being constructed in areas in which the future noise levels would approach or exceed NACs, while also providing an improved roadside appearance.

This measure, however, can become very expensive due to rising land costs and is almost never an option in areas that are already developed because the cost for acquiring already developed property (e.g., homes, businesses) is prohibitive.

Property owners cannot receive Federal funds as monetary compensation in lieu of noise abatement. FHWA regulations prohibit the use of Federal funds for such purposes, since they do nothing to reduce the noise levels or abate the highway noise impacts.

18.4.4 Noise Insulation

Insulation of buildings can greatly reduce traffic noise, especially when windows are sealed and cracks and other openings are filled. However, once windows are sealed, an air conditioning system will likely be necessary. New buildings can have sound absorbing material installed in the walls during construction. Noise insulation does nothing to improve the noise levels at adjacent outdoor use areas.

Federal funding can only be used for noise insulation for NAC D activity categories, which are listed in Table 18-1. If this option must be considered because no other feasible or reasonable mitigation measures are available, the condition of the structure, its amenities, and overall use characteristics must be thoroughly evaluated. Determinations such as these must be completely documented and are done on a case-by-case basis. Post installation maintenance and operational costs for noise insulation are not eligible for Federal-aid funding.

The only situation in which noise insulation would be considered for private dwellings is if extraordinary traffic noise impacts are found. Such a situation might exist where the projected noise levels are 75 dBA or greater or where the projected increase over existing levels is 30 dBA or more and no other possible abatement is reasonable and feasible. Under these conditions the project may use state and/or local agency funding to implement an insulation abatement solution only if the mitigation meets reasonable and feasible abatement criteria as is required for conventional noise mitigation. This determination must be made on a case-by-case basis.

18.5 NOISE BARRIERS

Construction of noise barriers is the most common noise abatement method. Essentially, a noise barrier is a solid structure that is constructed for the purpose of reducing noise levels. It may be a wall, a berm, or a combination of both. The barrier works by blocking the path of sound waves from the highway source, forcing it around or over the barrier. The incident sound wave is either reflected or absorbed by the barrier surface. Sound can also be transmitted through the barrier, which is why the barrier must be constructed without gaps and be sufficiently dense. Therefore, privacy fences do not function well as noise barriers.

Noise barriers are designed to reduce noise beginning at the first-row receptors, which are receptors closest to the barrier. Barriers may benefit receptors beyond the first row, depending on the configuration of the development. Normally, barriers are effective for receptors within

300 feet of the noise source if they are high enough to block the view of the roadway and are long enough to prevent sound from bending around the ends.

A noise reduction benefit of 5 dBA is generally fairly simple to achieve; however, a 7 dBA reduction is required at a minimum of one receptor along the wall in order for the barrier to be found reasonable. A reduction of 15 or more dBA is difficult to achieve, since it requires a reduction of at least 97 percent of the initial acoustic energy of the source.

18.5.1 Noise Barrier Walls

Noise barrier walls are a common means for reducing roadway noise levels. They can be constructed from a variety of materials. Although many wood walls were constructed in the past, life cycle and maintenance issues have resulted in the majority of new walls being constructed out of concrete, masonry block, or brick. The *CDOT Landscape Architecture Manual* (13) no longer allows wood to be used. Walls are preferred in many areas because they can be constructed where a limited amount of space precludes construction of an earth berm or a wall-berm combination.

Most noise walls are ground-mounted. In some situations, a barrier needs to be placed on a structure. This is most common for highway bridges, when the barrier needs to be constructed on the bridge to prevent a major gap in the barrier.

Illustration 18-1: Cast-in-Place Masonry I-76 East of York Street

Illustration 18-2: Post-and-Panel Noise Barrier Mounted on Type 7 Barrier: US-6 in Lakewood

18.5.2 Berms

Earth berms are a good alternative to noise barrier walls, since they have a more natural appearance and are aesthetically pleasing. Berms should be considered in areas where sufficient right of way is available to install them. This will help preserve the corridor visual and environmental qualities.

Feasibility of berm construction should be considered within the highway overall grading and drainage plan, particularly if an irrigation system will be part of the project. One advantage of berm construction is that a variety of materials, such as soil, stone, rock, or rubble can be used. Typically, berms can be constructed from surplus material available directly on the project site or from waste material from other areas. This can result in decreased costs compared to the cost of a noise barrier.

Slopes of an earth berm should be 2:1 or flatter, for safety and erosion control purposes, although a 3:1 slope is preferable. The ends of the berm should have a lead-in slope of 10:1 and curve toward the highway. Berms can be either vegetated or seeded. Slope stabilization should be done as soon as possible after construction.

18.5.3 Combination Barriers

For areas where a full berm cannot be constructed, such as a situation where there is limited right of way, a combination barrier can be constructed. This involves building up the earth berm to a desired height and constructing a wall on the berm. The soil in the berm must be stable enough to support a wall structure foundation.

Illustration 18-6: Recycled Sand Berm: I-70 East of Vail

Illustration 18-7: Combination Barrier: I-25 @ Exit 188

18.5.4 General Design Guidelines

The following are general considerations for noise barrier design:

- Barriers should not be installed where they will present a safety hazard. A desirable location is just inside of the right of way line or outside the clear zone. If a barrier needs to be located inside of the clear zone, a guardrail or other traffic barrier may be warranted and the barrier material should meet minimum impact standards to prevent shattering. Light reflected to motorists should be minimized.
- A barrier should not block the line-of-sight between a vehicle on a ramp and approaching vehicles on a major roadway. For entrance and exit ramps, ramp intersections and intersecting roadways, the proper barrier location should be determined based on stopping distance requirements. Barrier end points should be approved by the CDOT Region Traffic Engineer.
- Barriers which are oriented in an east-west fashion and have a long barrier face should have the shadow cast checked for encroachment into the shoulder or near traffic lane. Since barriers obstruct light as well as noise, special consideration should be given to icing or other environmental conditions caused by the barrier placement. This consideration should also be given to shadow coverage in adjacent yards and parking lots. This should not be an issue with barriers that are oriented north-south.

- Protrusions on barriers near traffic lanes or facings, which can become missiles in a crash, or which can create excessive glare, should be avoided.
- Positive mechanical connection of the individual noise barrier panels to the posts is required when the noise barrier is on a bridge or retaining wall in the vicinity of pedestrian or vehicular traffic or immediately adjacent to private property.
- Provisions may be necessary to allow firefighters or HAZMAT crews access to fire hydrants on the opposite side of the barrier. This should be coordinated with the appropriate jurisdictional entity.
- Drainage considerations need to be taken into account to ensure soil stability.
- For noise wall structural design considerations, refer to the AASHTO *Guide Specifications for Structural Design of Sound Barriers.* **(14)** Some structural aspects to consider on a project –specific basis are: Can the barrier be easily mounted to a bridge? Can it be retrofitted in the future? Does it accommodate through-the-wall access doors? Is it capable of supporting signs or lighting? It is preferred that signs and lighting have their own foundations.
- Barriers should be integrated with other project elements, such as foundations impacts on underground facilities and barrier impacts to overheat facilities.

Project environmental documents or noise analysis studies, if available, specify recommendations regarding general locations, noise reductions, barrier heights, and barrier lengths. These are some of the considerations that are taken into account when the acoustical analyst arrives at barrier recommendations:

- The barrier should be high enough and long enough to cause an effective "sound shadow zone" for the adjacent receptors. Receptors located within the shadow zone do not have direct line-of-sight to the noise source (highway).
- The barrier location should take advantage of local terrain conditions to benefit from higher elevations if possible.
- Normally, the noise barrier should not exceed a height of 20 feet above the traveled way, nor should it be shorter than 8 feet. If the barrier is constructed on the shoulder, 12 feet is a recommended maximum height. Special geographic considerations, however, may warrant taller walls or allow a shorter wall to provide the desired noise reduction.
- The design plans should always indicate the top and bottom elevations of the barrier.

- The relationship between the height of the barrier and its noise reduction characteristics is not linear. As a rule, a barrier breaking the line-of-sight will provide a 5-dBA reduction, with an additional 1-dBA reduction resulting with each additional 2 feet in height. At the receptor end, the line-of-sight is always checked from a point 5 feet above the ground elevation, which approximates the height of the average human ear.
- Building the barrier closer to either the receptor or the noise source provides more noise reduction compared to locating the barrier in the middle between the receptor and source. However, this is not practical in all cases.
- To prevent noise from flanking around the barrier ends, the barrier should extend past the end receptor at least four times the perpendicular distance from the receptor to the barrier. If this is not possible, the barrier can be bent back towards the receptor (wrapping the barrier) in order to reduce noise at the receptor. Also, combining the barrier with natural terrain features and existing structures may help reduce noise at the receptors.
- When barriers are placed opposite each other on different sides of the same highway (parallel barriers), there is the possibility for degradation of the performance of the barrier system if the width-to-height ratio (distance between the barriers vs. the barrier height) is 10:1 or less due to multiple reflections. In these cases, raising the barrier heights or providing absorptive treatments may need to be considered.
- Noise absorption (materials or treatments) should be considered for single highway barriers that have the potential to reflect noise into unprotected areas.
- Gaps in the noise barrier can significantly degrade barrier performance. These include breaks for structures, drainage ditches, emergency accesses, frontage roads, driveways, and ramps. If barrier gaps are inevitable, degradation in the barrier performance can be reduced by providing tight fitting access doors, using small openings for drains and culverts, wrapping the barrier back toward the receptors, or overlapping two barrier segments. If overlapping barriers are used, the length of the overlap should be at least four times the width of the gap or opening to prevent any further degradation in the barrier's performance.

18.5.5 General Aesthetic Guidelines

The visual impact on adjacent land users is a consideration in noise barrier design. A primary factor is scale relationship between the noise barrier and activities adjoining the highway right of way. A high noise barrier alongside low, single-family residences could have a severe visual effect or create adverse shadows that affect property and landscaping by reducing overall sunlight. In general, a barrier should be at least four times its height from residences to prevent alteration of residential microclimates and the area between should be landscaped. Due to all of the issues which must be addressed when considering noise barrier construction, an interdisciplinary team of highway engineers, structural engineers, noise analysts, environmental personnel, and landscape architects should be formed to provide specialized input and expertise early on in the process. Additional information regarding aesthetic guidelines for noise barriers can be found in the *CDOT Landscape Architecture Manual*. **(13)**

18.5.5.1 Visual Analysis

An important step in the design process is the visual analysis of the proposed site and consideration of relationships that occur between the neighborhood, community, and geographical area for which the design is intended. Community context and passing motorists' perspective should be documented in the visual aesthetics assessment. Ultimately, the proposed design will become a part of the neighborhood or community. Local public involvement should be incorporated into design of visually aesthetic elements to gain input and feedback from the affected community stakeholders. Development of visual design guidelines should be used as a planning tool to emphasize visual quality, continuity, and consistency for transportation corridors with multiple noise wall projects.

One of the key elements of a visual analysis is character. Each community has a distinct character; thus, a visual analysis should include a determination of the neighborhood's character. The site and surroundings should be classified into rural, urban, and suburban categories. Each of these categories has unique environmental and social characteristics that should be considered for visual design. Noise barriers should be carefully considered in relationship to the setting so, when possible, they reflect the community's character and neighborhood's style.

The character of a rural environment is one of open spaces, native trees, shrubs, grasses, and earth and sky which promote a random, natural appeal. Noise barriers in these areas should be constructed so that they will appear to be associated with the rural atmosphere. The adjacent area should be planted with native plant materials in random groupings. In contrast, the urban environment is one of geometric lines, orderly development, human activity, confined spaces, structures, and pavement. Random, natural groupings of plant materials can be used to complement noise barriers in urban areas where a community desires to make a statement about its image, connection, and support of the natural environment.

Illustration 18-3: Artistically Treated Precast Concrete Noise Barrier: SH-47 in Pueblo

Visual analysis and landscape inventory data is useful at public meetings and neighborhood workshops where citizens can be offered a choice of wall type, materials, colors, and textures. Many objections from the public relate to a loss of scenic views or to the visual appearance of the noise barrier. A barrier is more likely to be accepted by the public if it is a visual complement to the community.

Including community leaders and representatives in the design process enables them to share their ideas and discuss how local character might be incorporated into the noise wall design. This process is described in the Chief Engineer's Policy Memo 26 (Context Sensitive Solutions [CSS] Vision for CDOT). Citizen participation in the design process results in fewer post-construction complaints.

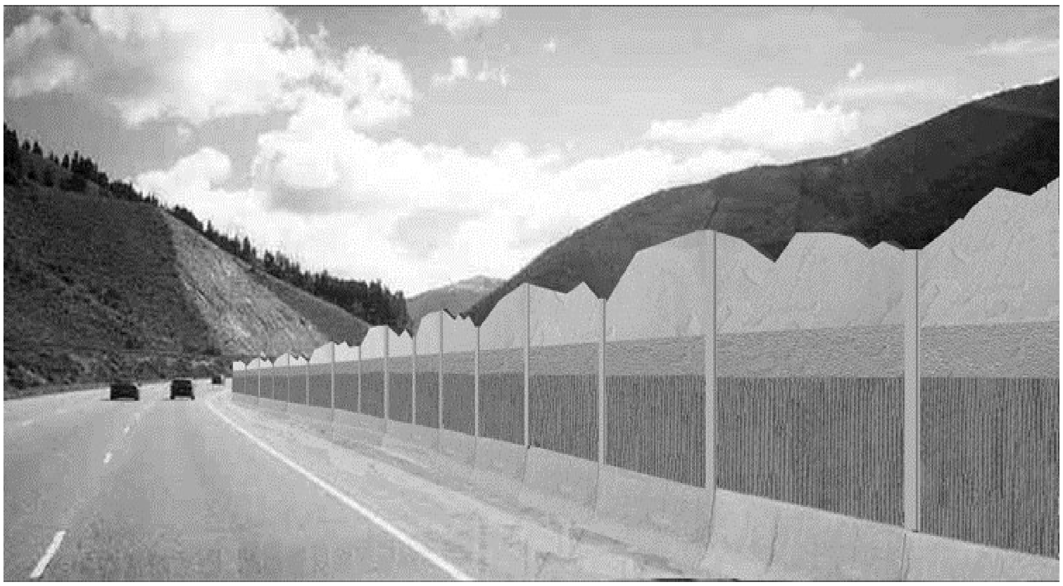
18.5.5.2 Visual Design Principles

Visual quality is a product of the design process, and a visually attractive wall can be built without excessive additional expense. In general, a highway setting with a noise barrier can be described as an enclosed space, which can result in negative reactions by motorists traveling on the roadway or residents who border the barrier. Several design principles can be applied to the noise barrier design that reduce the effect an enclosure has on the senses:

- LINE. A line is a direct connection of two points and can be either real or implied. Long straight lines, like those associated with noise barriers, rarely occur in nature. As a result, the barrier "line" tends to dominate the surrounding area. Techniques that can be used to de-emphasize long lines are placement of horizontal and vertical lines in proper locations, use of curves, altering the vertical locations of wall sections, and use of lines to reflect skylines, buildings, or natural landforms such as rivers or mountains.
- FORM. Form exists in nature as mountains, boulders, or landforms. Separate objects can create forms when viewed from a distance or at highway speeds such as several trees forming a grove or many buildings forming a single skyline.
- COLOR. Color is the breakdown of light into individual visual elements. Color reactions are intuitive in most people. Generally, bright and vivid color combinations, such as those used in billboards or commercial signage, produce startling and sometimes aggressive effects. Noise barriers can use more neutral, subdued colors like those found in natural materials to create a more soothing effect. **(13)**
- TEXTURE. Texture usually refers to varying degrees of coarseness or smoothness of an object's surface. Whether natural or manmade, texture is a factor in the visual interest of the surrounding area. Smooth surfaces tend to blend into monotone, making them hard to differentiate. They also may reflect noise, which is discouraged. Coarser surface textures can be seen and recognized from greater distances. Coarse surface wall texture may be a slight deterrent to graffiti artists. **(13)**
- CONTRAST: The natural environment is generally of low to medium contrast. High contrast is desired where it is important that objects stand out, such as pavement markings and road signs, but it may be detrimental when trying to blend a noise barrier into its surroundings.
- SEQUENCE. Sequence is a progression of the visual experience of movement or change. This principle can be used to visually link one event with another to direct the eye to a desired point. Sequences in noise barriers can be created by selective end treatments, repeating attractive landscape planting groups, or repeating patterns in noise wall panels.
- AXIS. Axis is a visible or invisible line that divides a view. Most axes in nature are asymmetrical, while in manmade environments they tend to divide a view into equal parts. Symmetrical axes tend to be monotonous. The proper use of both concepts is desirable for long stretches of highway, which is normally a monotonous setting.
- DOMINANCE. Dominance refers to a comparison between adjacent objects in terms of visual importance. Since they are usually large structures, noise barriers tend to dominate

areas in which they are placed. This is a result of the difference in scale between a noise barrier and the typical human environment.

Illustration 18-4: I-70 Dillon Valley Noise Barrier
(Photo Simulation Courtesy of PKM Design Group)



18.5.5.3 End Treatments

Noise walls should appear to be part of the existing landscape and not give the impression that they were placed as an afterthought. Noise walls should begin and end in a natural transition, if possible, from the ground level to the desired height.

Where space allows, the best transition is a natural earth berm or terrain feature in which the end of the wall can be incorporated. Through this technique, the wall appears to originate from the landscape rather than be dropped onto it. If there are no terrain features in the area, a "step-down" technique at the end of the barriers can provide the same effect. Any tapering of the wall should be gradual to a point where the wall is no longer visually dominant. Walls should tie into and match aesthetics of existing structures, such as bridges, bridge abutments, and retaining walls.

18.5.5.4 Design Elements for Landscape Plantings

Landscape plantings may be the most effective and economical means available to reduce adverse visual impacts of a noise barrier. When used in combination with a structure, plantings serve to link the structure of the wall with the natural surroundings. Trees, shrubs, and grasses may provide all of the design elements of line, form, color, and texture and mitigate problems with scale and dominance of the noise wall in the landscape. They also provide a living, changing element in a hard-edged manmade environment and provide psychological and visual relief to adjacent communities.

Massing shrubs and trees can create a natural transition area for the end of a wall. Plant materials can provide color and texture variety and can have positive effects on scale and dominance. Vegetation can also provide shade, reduce reflection of noise and light, cool and filter the highway environment, provide slope stabilization, and reduce erosion control problems.

Following are some landscape plantings design guidelines:

- Design planting “pockets” by creating offsetting recesses within the noise wall line. Small jogs can provide protected microclimates and visually soften wall impacts on the motorist.
- Use vines and shrubs in combination to reduce the dominance of a wall.
- Vary heights and textures to get a good combination of plant masses.
- Vary spacing and tree groupings to improve the visual effect, where possible.
- Use trees to reinforce rhythm and sequence and provide a vertical element in predominantly horizontal walls.
- Use ground covers and shrubs to smooth the transition between the wall and ground plane.
- Install plantings against the wall and opposite the wall to provide an asymmetrical axis and reduce visual dominance.
- Use plantings that are drought tolerant and relatively maintenance free. Select plant species from regional and local reference sources that document their hardiness, microclimate appropriateness, and relative longevity.
- Use good planting practices, such as appropriate soil amendments, groupings of plants with like water needs, and low-cost, attractive mulches to conserve water and moderate summer and winter soil temperatures.

Illustration 18-5: Terraced Landscaping With Noise Barrier: US-287 Lafayette Bypass

18.5.6 General Maintenance Guidelines

When considering construction of a noise barrier, maintenance factors should be addressed and any fatal flaws identified as early as possible to prevent problems in either design or operation. Examples of these factors include maintenance of the barrier, protective coatings, replacement of materials damaged by impact, cleaning of the barrier, graffiti prevention and removal, snow storage, and de-icing of the roadway in the winter months if shadowing is a problem. Plantings should be tolerant of the roadside environment and require little to no maintenance. It is particularly important to maintain a stock of replacement materials (i.e., posts, panels, blocks), which are compatible with the barrier in case damage does occur. Additional quantities should be considered in the construction package for contingency purposes.

Access to the barrier backside is usually needed. Access can be provided with an access road, a walk path, gates, or access panels built into the barrier. Access must be designed so that it does not compromise the noise reduction effectiveness of the barrier. If the barrier is constructed on the right of way line, provisions should be made to coordinate the location of the access points with the appropriate agencies or landowners.

18.6 General Materials Guidelines for Noise Barrier Walls

To ensure that all materials used to build noise barrier walls meet acoustic requirements, material information and test results must be submitted to the CDOT Product Evaluation Coordinator for approval to be added to CDOT's Approved Products List. To be approved, the material must meet testing requirements, as described in Section 18.6.1. CDOT also evaluates materials based on additional criteria for which testing methods do not apply, as described in Section 18.6.2.

CDOT may request to view, in person, a sample or a full size section of the barrier product, at CDOT's discretion. Tests shall be performed by a certified independent third party. To obtain valid results, specimens that get tested should be taken from a finished production run product and not from small handmade pieces that were specifically made to be tested.

18.6.1 Acoustic Testing Requirements

- Materials shall have a minimum acceptable Sound Transmission Class (STC) of 30, as tested using ASTM E90 and ASTM 413 or a CDOT approved equivalent specification.

- Materials shall have a minimum Noise Reduction Coefficient (NRC) of 0.70 if seeking an "absorptive" classification, as tested using ASTM C423 or a CDOT approved equivalent specification. Materials that are not tested or do not meet this requirement shall be classified as "reflective."

18.6.2 Additional Considerations for Noise Barrier Materials

18.6.2.1 Acoustic Properties

- Materials must be acoustically durable over the design life. Absorptive surface treated walls must resist degradation of sound-absorbing properties after installation. The materials should not require cleaning in order to maintain sound-absorbing properties.

- Project plans should indicate if the noise wall surface is reflective or absorptive

18.6.2.2 Physical Properties

- Noise wall materials of concrete panels, masonry blocks, or brick are used most frequently because of their life cycle cost and maintenance considerations. Noise walls are generally built using concrete and concrete durability properties and coating properties for concrete are not unique to noise walls. If noise walls are designed with another material, durability and coating properties would be examined on a case-by-case basis. The CDOT Landscape Architecture Manual (14) does not allow use of wood.

- Barriers shall be designed and constructed without gaps, or, if an opening is required, the gap shall be minimalized.

- Generally, barrier heights are a minimum of 8 feet and a maximum of 20 feet. For barriers constructed on the shoulder, 12 feet is a recommended maximum height. Project design may adjust these dimensions if required.

- Privacy fences rarely have the acoustic properties to function as noise barriers.

18.6.2.3 Maintenance Requirements

- Materials must be resistant to impact or easily replaceable or repairable using CDOT-owned equipment.

- Surface texture, coating, or combination thereof of walls in areas subject to graffiti should make the graffiti difficult to place and easy to remove. Details of the process to remove graffiti should be provided to CDOT.

A list of pre-approved Absorptive and Reflective Sound Walls is available on CDOT's Approved Products List at www.codot.gov/business/apl.

18.7 CONSTRUCTION NOISE

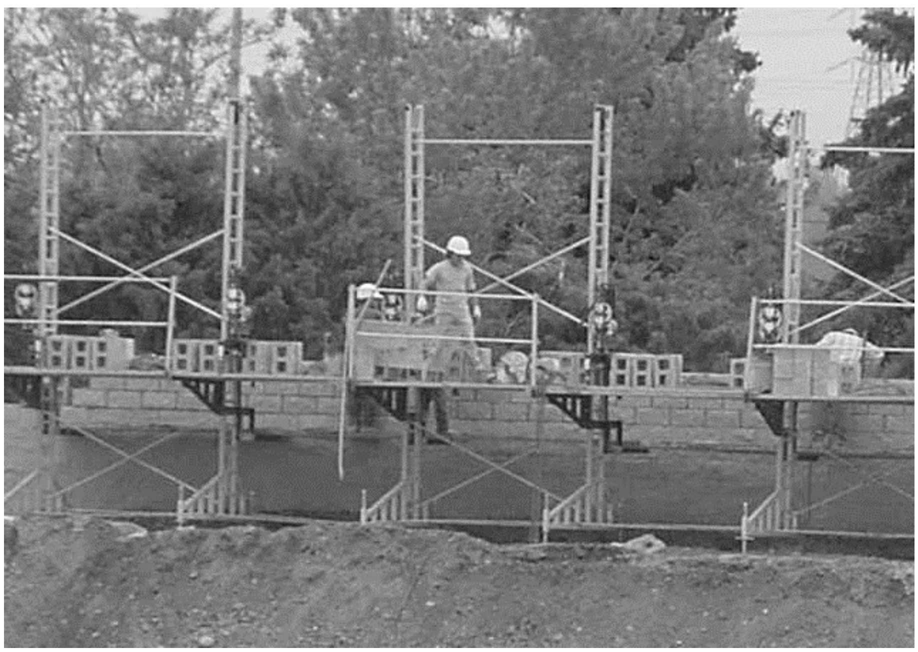
The approach to construction noise should be general in scope and consider the temporary nature of construction activities. Although the public generally views construction noise as a short-term issue that is tolerable and necessary, types of activities that are expected to be performed and equipment that will be used should be disclosed.

Although a detailed analysis of mitigation measures is not generally required, the noise analysis should at least identify low-cost, practical mitigation measures that can be included on the project. Examples are limiting work to daytime hours, ensuring that equipment uses properly maintained mufflers, modification of backup alarm systems, location of haul roads, construction of feasible and reasonable noise barriers as soon as possible, and public outreach. Noise mitigation may be a larger issue on large, complex projects in major urban areas. For these projects, a more detailed discussion is necessary and may require a separate report detailing monitoring and mitigation measures.

Some local government agencies have local noise ordinances that restrict how much noise may be emitted during certain hours or in certain areas (e.g., residential neighborhoods). These noise ordinances must be obeyed unless a variance has been approved. Such a variance may be needed if the work will be very extensive or lengthy.

For additional assistance, refer to the following references or contact the CDOT Noise Program Manager at: http://www.coloradodot.info/programs/environmental/noise.

Illustration 18-8: Masonry Noise Barrier Construction: I-270 West of York Street



REFERENCES

Updated versions of various FHWA references also found at http://www.fhwa.dot.gov/environment/noise/

1. Code of Federal Regulations (CFR), Title 23, Part 772: Procedures for Abatement of Highway Traffic Noise and Construction Noise, revised July 2010.
2. *CDOT Noise Analysis and Abatement Guidelines*, January 15, 2015.
3. FHWA. *Entering the Quiet Zone: Noise Compatible Land Use Planning*, Washington, D.C.: Federal Highway Administration, U.S. Department of Transportation, May 2002. http://www.fhwa.dot.gov/environment/noise/noise_compatible_planning/federal_approach/land_use/qz00.cfm
4. FHWA. *Highway Noise Barrier Design Handbook*, Federal Highway Administration, U.S. Department of Transportation, Washington, D.C.: February 2000.
5. FHWA. *Highway Construction Noise Handbook*, Federal Highway Administration, U.S. Department of Transportation, Washington, D.C.: 2006.
6. FHWA *Highway Noise: A Guide to Visual Quality in Noise Barrier Design*, Federal Highway Administration, U.S. Department of Transportation, Washington, D.C.: , April 1994.
7. FHWA. *Highway Traffic Noise: Analysis and Abatement Guidance*, Federal Highway Administration, U.S. Department of Transportation, Washington, D.C.: December 2011.
8. FHWA. *Insulation of Buildings Against Highway Noise*, Federal Highway Administration, Washington, D.C.: 2011.
9. FHWA. Lee, Cynthia S.Y., Gregg G. Fleming. *Measurement of Highway-Related Noise*. Report No. FHWA-PD-96-046 and DOT-VNTSC-FHWA-96-5. Cambridge, MA: John A. Volpe National Transportation Systems Center, Acoustics Facility, May 1996.
10. FTA. *Transit Noise and Vibration Assessment,* Federal Transit Administration, May 2006.
11. CDOT Procedural Directive 1601, "Interchange Approval Process," October 16, 2008.
12. *CDOT Environmental Stewardship Guide,* May 2005.
13. CDOT *Landscape Architecture Manual*, August 18, 2014
14. AASHTO. *LRFD Bridge Design Specifications, Customary U.S. Units, 6th Edition, with*

2013 Interim Revision, Washington, D.C.: American Association of State Highway and Transportation Officials, 2013. [Content formerly included in AASHTO *Guide Specifications for Structural Design of Sound Barriers*.]

CHAPTER 19
ROUNDABOUTS

19.0 INTRODUCTION

A roundabout is a form of a circular intersection in which traffic travels counterclockwise around a central island where entering traffic must yield to circulating traffic. Roundabouts feature, among other things, a central island, a circulatory roadway, and splitter islands on each approach. Roundabouts rely upon two basic and important operating principles:

1. Speed reduction at the entry and through the intersection will be achieved through geometric design and,

2. The yield-at-entry rule, which requires traffic entering the intersection to yield to traffic that is traveling in the circulatory roadway.

Benefits of roundabouts are:

- Fewer conflict points typically result in fewer collisions with less severity. Over half of vehicle to vehicle points of conflict associated with intersections are eliminated with the use of a roundabout. Additionally, a roundabout separates the points of conflict which eases the ability of the users to identify a conflict and helps prevent conflicts from becoming collisions.
- Roundabouts are designed to reduce the vehicular speeds at intersections. Lower speeds lessen the vehicular collision severity. Likewise, studies indicate that pedestrian and bicyclist collisions with motorized vehicles at lower speeds significantly reduce their severity.
- Roundabouts allow continuous free flow of vehicles and bicycles when no conflicts exist. This results in less noise and air pollution and reduces overall delays at roundabout intersections.

The following is a list of locations where a roundabout may be feasible:

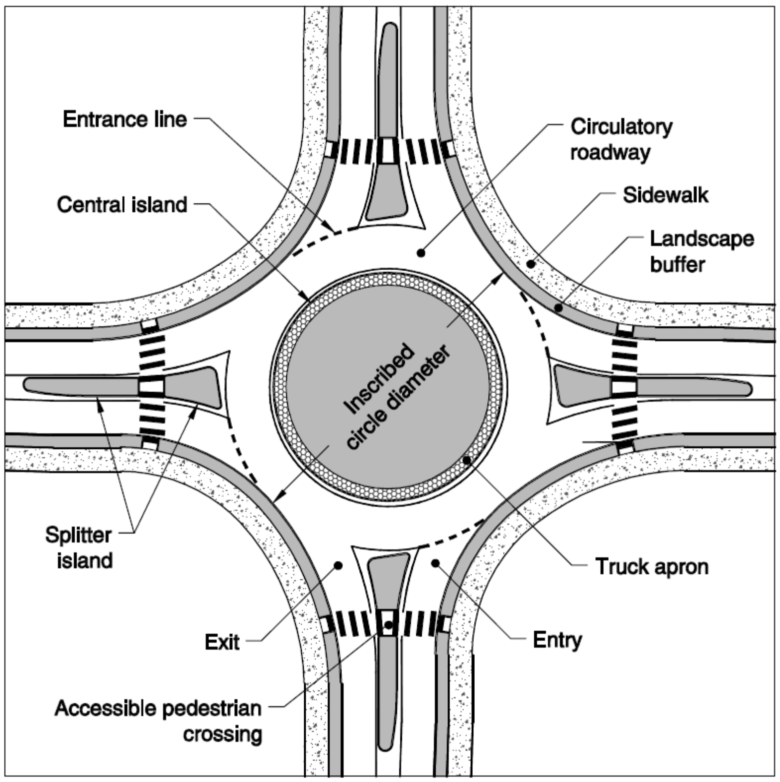
- Intersections with a high-crash rate or a higher severity of crashes
- High-speed rural intersections
- Freeway ramp terminals
- Transitions in functional class or desired speed change (including rural to urban transitions)
- Existing intersections that are failing operationally
- Intersections where aesthetics is an objective
- Four-leg intersections with entering volumes less than 5,000 vph or approximately 50,000 ADT
- Three-leg intersections
- Intersection of two signalized progressive corridors where turn proportions are heavy (random arrival is better than off-cycle arrival)
- Closely spaced intersections where signal progression cannot be achieved
- Replacement of all-way stops
- Intersections near schools

The contents of this chapter are intended to serve as design guidance only.

Roundabout intersections on the Colorado State Highway System must be developed and evaluated in accordance with National Cooperative Highway Research Program (NCHRP) Report 672 (1) entitled "Roundabouts: An Informational Guide, 2nd ed." (NCHRP Guide 2) dated October 2010, or latest edition.

Roundabout considerations, planning, operational analysis and safety are not covered in this chapter. Signs, striping and markings at roundabouts are to comply with the MUTCD latest edition.

Figure 19-1 depicts the typical nomenclature associated with roundabouts.



Note: This figure is provided to only shown nomenclature and is not to be used for design details.

Figure 19-1 [NCHRP Report 672 Exhibit 6-1 (1)] Roundabout Geometric Elements

19.1 ROUNDABOUT CATEGORIES

Roundabouts are separated into three basic categories according to the size and number of lanes used at the roundabout. The three categories of roundabouts are: mini-roundabouts, single-lane roundabouts, and multilane roundabouts. Table 19-1 summarizes and compares some fundamental design and operational elements for each roundabout category.

Design Element	Mini-Roundabout	Single-Lane Roundabout	Multilane Roundabout
Desirable maximum entry design speed	15 to 20 mph (25 to 30 km/h)	2 to 25 mph	25 to 30 mph
Maximum number of entering lanes per approach	1	1	2+
Typical inscribed circle diameter	45 to 90 ft	90 to 180 ft	150 to 300 ft
Central island treatment	Fully traversable	Raised (may have traversable apron)	Raised (may have traversable apron)
Typical daily service volumes on 4-leg roundabout below which may be expected to operate without requiring a detailed capacity analysis (veh/day)*	Up to approximately 15,000	Up to approximately 25,000	Up to approximately 45,000 for two-lane roundabout
*Operational analysis needed to verify upper limit for specific applications or for roundabouts with more than two lanes or four legs.			

Table 19-1 [NCHRP Report 672 Exhibit 1-9 (1)] Roundabout Category Comparison

19.1.1 Mini-Roundabout

Mini-roundabouts are small single-lane roundabouts with a fully traversable central island. They are most commonly used in low-speed urban environments with average operating speeds of 30 mph or less. Figure 19-2 illustrates a typical mini-roundabout and the important characteristics. Mini-roundabouts can be useful in such environments where conventional roundabout design is precluded by right-of-way constraints. In retrofit applications, mini-roundabouts are relatively inexpensive because they typically require minimal additional pavement at the intersecting roads and minor widening at the corner curbs. They are mostly recommended when there is insufficient right-of-way to accommodate the design vehicle with a traditional single-lane roundabout. Because they are small, mini-roundabouts are perceived as pedestrian-friendly with short crossing distances and very low vehicle speeds on approaches and exits.

A fully traversable central island is provided to accommodate large vehicles and serves one of the distinguishing features of a mini-roundabout. The mini-roundabout is designed to accommodate passenger cars without requiring them to traverse over the central island. The overall design of a mini-roundabout should align vehicles at entry to guide drivers to the intended path and minimize running over of the central island to the extent possible.

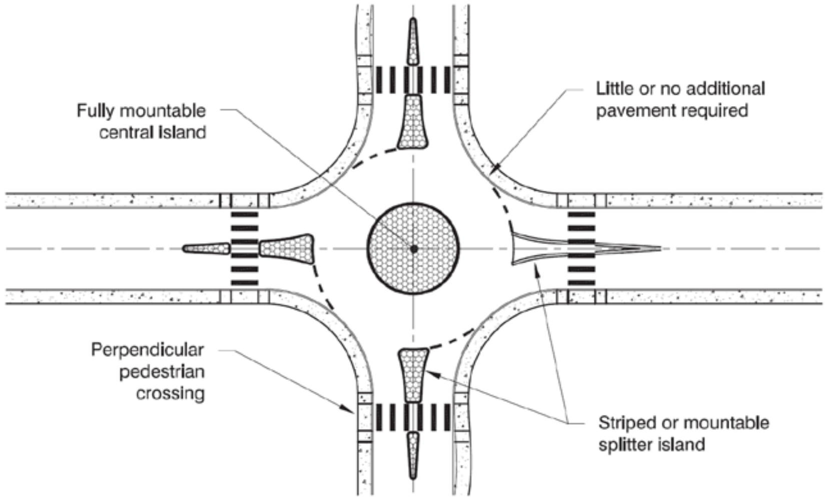


Figure 19-2 [NCHRP Report 672 Figure 1-10 (1)] Typical Mini-Roundabout

19.1.2 Single-Lane Roundabout

This type of roundabout is characterized as having a single-lane entry at all legs and one circulatory lane. Figure 19-3 illustrates the features of a typical single-lane roundabout. They are distinguished from mini-roundabouts by their larger inscribed circle diameters and non-traversable central islands. Their design allows slightly higher speeds at the entry, on the circulatory roadway, and at the exit. The geometric design typically includes raised splitter islands, a non-traversable central island, crosswalks, and a truck apron. The size of the roundabout is largely influenced by the choice of design vehicle and available right-of-way.

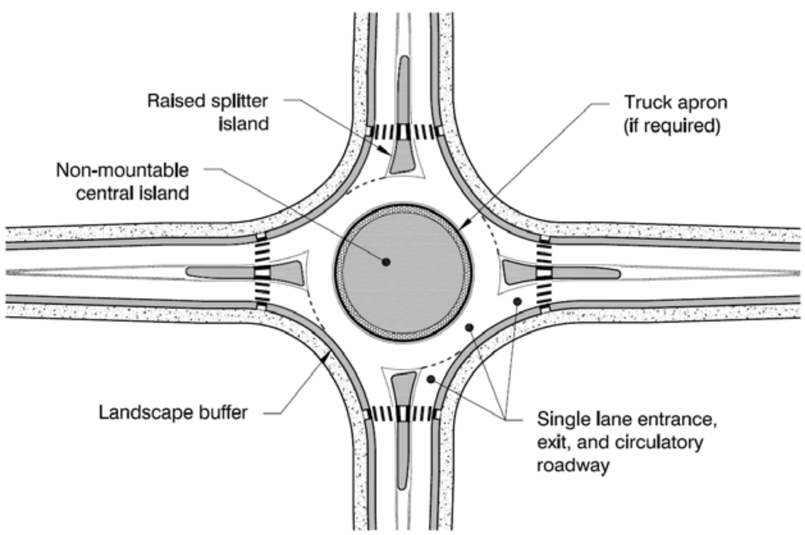


Figure 19-3 [NCHRP Report 672 Figure 1-12 (1)] Typical Single-Lane Roundabout

19.1.3 Multilane Roundabout

Multilane roundabouts have at least one entry with two or more lanes. In some cases, the roundabout may have a different number of lanes on one or more approaches. They also include roundabouts with entries on one or more approaches that flare from one to two or more lanes. These require wider circulatory roadways to accommodate more than one vehicle traveling side by side. Figure 19-4 provide an example of a typical multilane roundabout. The speeds at the entry, on the circulatory roadway, and at the exit are similar or may be slightly higher than those for the single-lane roundabouts. The geometric design will include raised splitter islands, truck apron, a non-traversable central island, and appropriate entry path deflection.

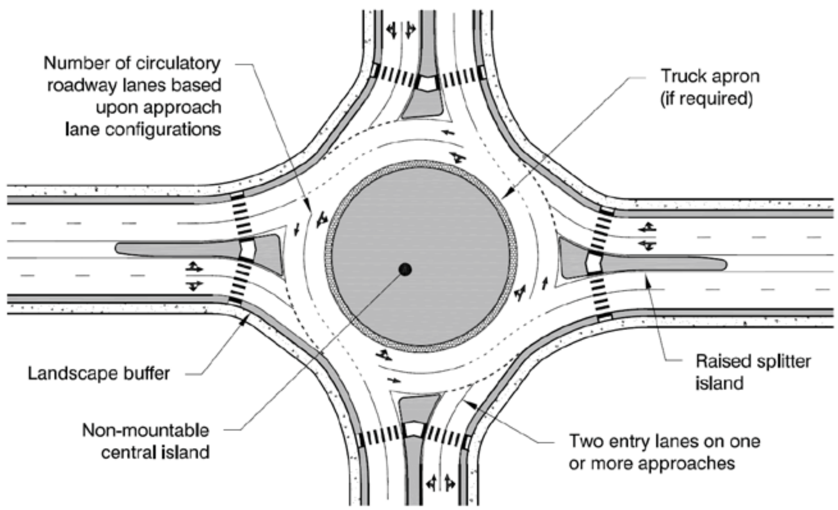


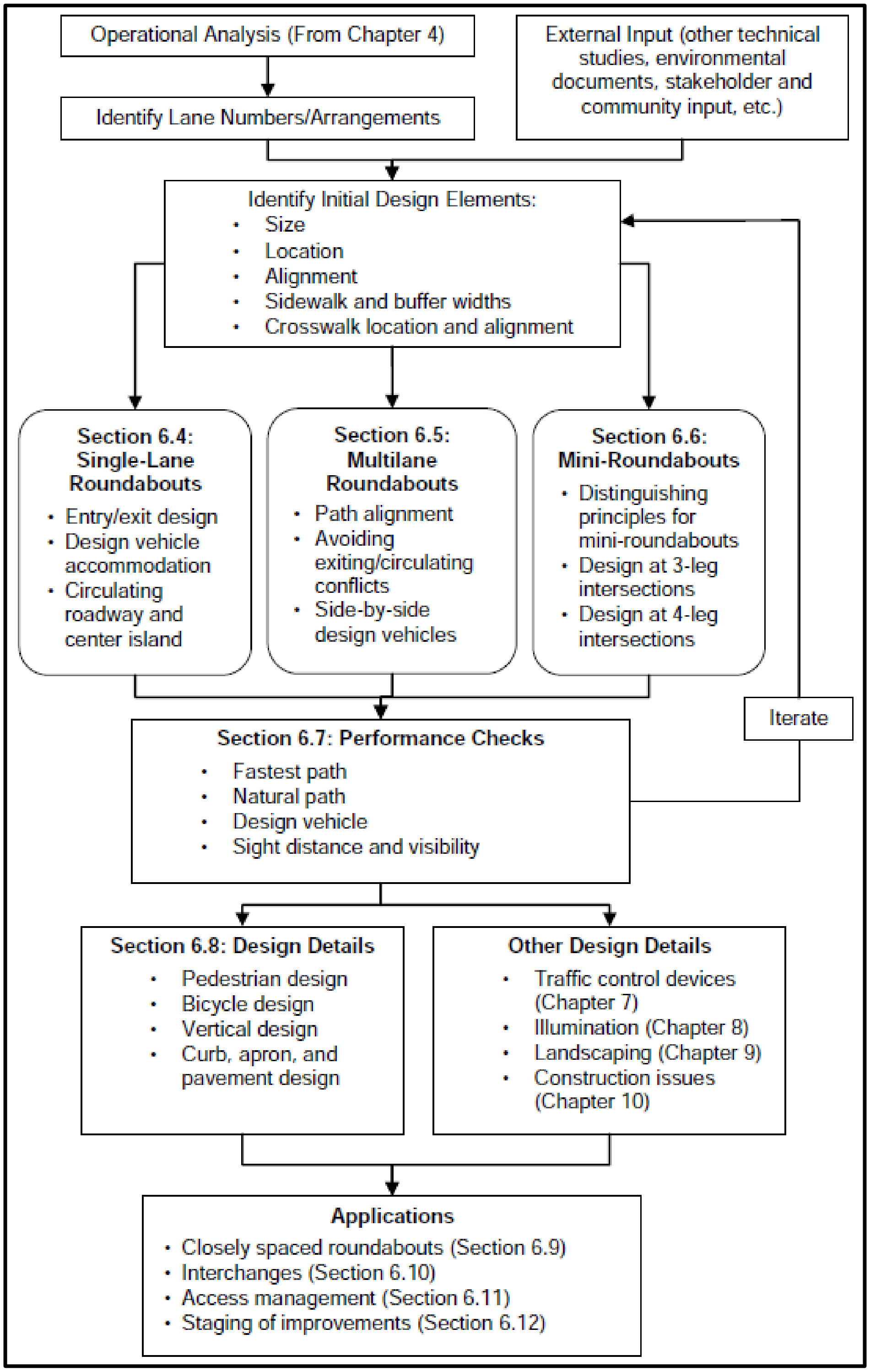
Figure 19-4 [NCHRP Report 672 Figure 1-14 (1)] Typical Multilane Roundabout

19.2 ROUNDABOUT DESIGN PROCESS

19.2.1 Roundabout Design Process

Roundabout design is an iterative process where a variety of design objectives must be considered and balanced within site-specific constraints. Maximizing the operational performance and safety for a roundabout requires the engineer to think through the design rather than rely upon a design template. The basic design should be laid out based upon the principles to a level that allows the engineer to verify that the layout will meet the design objectives. The key is to conduct enough work to be able to check the design and identify whether adjustments are necessary. Once enough iteration has been performed to identify an optimum size, location, and set of approach alignments, additional detail can be added to the design.

Figure 19-5 provides a general outline for the roundabout design process, incorporating elements of project planning, preliminary design, and final design into an iterative process. Information from the operational analysis is used to determine the required number of lanes for the roundabout (single or multilane), which dictates the required size and many other design details. The basic design should be laid out based upon the principles identified in this chapter and the *NCHRP Report 672* to a level that allows the engineer to verify that the layout will meet the design objectives.



Note: Section numbers refer to NCPRP Report 672

Figure 19-5 [NCHRP Report 672 Exhibit 6-1 (1)] General Roundabout Design Process

19.2.2 General Design Considerations

Throughout this chapter and the *NCHRP Report 672* **(1),** ranges of typical values are given for many of the different geometric elements to provide guidance in the design of individual roundabout components. The use of a design technique not explicitly included or a value that falls outside of the ranges presented does not automatically create an unsafe condition if a few basic design principles can be achieved. The following list of principles should be the objective of any roundabout design:

- Provide slow entry speeds and consistent speeds through the roundabout by using deflection.
- Provide the appropriate number of lanes and lane assignment to achieve adequate capacity, lane volume balance, and lane continuity.
- Provide smooth channelization that is intuitive to drivers and results in vehicles naturally using the intended lanes.
- Provide adequate accommodation for the design vehicles.
- Design to meet the needs of pedestrians and cyclists.
- Provide appropriate sight distance and visibility for driver recognition of the intersection and conflicting users.

Each of the principles described above affects the safety and operations of the roundabout. When developing a design, the trade-offs of safety, capacity, cost, and so on must be recognized and assessed throughout the design process. Favoring one component of design may negatively affect another. A common example of such a trade-off is accommodating large trucks on the roundabout approach and entry while maintaining slow design speeds. Increasing the entry width or entry radius to better accommodate a large truck may simultaneously increase the speeds that vehicles can enter the roundabout. Therefore, the engineer must balance these competing needs and may need to adjust the initial design parameters.

19.3 GEOMETRIC DESIGN

The following geometric design elements are a general set of guidelines to be considered when first laying out a roundabout. These are not to be interpreted as a standard or rule, but general best practices. As described above, roundabout design is an iterative process where a variety of design objectives must be considered and balanced within site-specific constraints.

19.3.1 Identify Initial Design Elements

19.3.1.1 Roundabout Size

The inscribed circle diameter is the overall outside diameter of a roundabout, which is the distance across the circle inscribed by the outer curb (or edge) of the circulatory roadway, as illustrated previously in Figure 19-1. The inscribed circle diameter is determined by a number of design objectives, including accommodation of the design vehicle and providing speed control.

The inscribed circle diameter typically needs to be at least 105 feet to accommodate a WB-50 design vehicle. Smaller roundabouts can be used for some constrained urban intersections, where the design vehicle may be a bus or single-unit truck. For locations that must accommodate a larger WB-67 design vehicle, a larger inscribed circle diameter will be required, typically in the range of 130 to 150 feet. In situations with more than four legs, larger inscribed circle diameters may be appropriate. Truck aprons are typically needed to keep the inscribed circle diameter reasonable while accommodating the larger design vehicles. Generally, the inscribed circle diameter of a multilane roundabout ranges from 150 to 250 feet. For two-lane roundabouts, a common starting point is 160 to 180 feet. Roundabouts with three- or four-lane entries may require larger diameters of 180 to 330 feet to achieve adequate speed control and alignment. Mini-roundabouts serve as a special subset of roundabouts and are defined by their small inscribed circle diameters. With a diameter less than 90 feet, the mini-roundabout is smaller than the typical single-lane roundabout. The small diameter is made possible by using a fully traversable central island to accommodate large vehicles.

Table 19-2 provides typical ranges of inscribed diameters for various roundabout configurations.

Roundabout Configuration	Typical Design Vehicle	Common Inscribed Circle Diameter Range*
Mini-Roundabout	SU-30	45 to 95ft
Single-Lane Roundabout	B-40	90 to 150 ft
	WB-50	105 to 150 ft
	WB-67	130 to 180 ft
Multilane Roundabout (2 lanes)	WB-50	105 to 220 ft
	WB-67	165 to 220 ft
Multilane Roundabout (3 lanes)	WB-50	200 to 250 ft
	WB-67	220 to 300 ft
* Assumes 90° angles between entries and no more than 4 legs. List of possible design vehicles in not all-inclusive.		

Table 19-2 [NCHRP Report 672 Exhibit 6-9 (1)] Typical Inscribed Diameter Ranges

19.3.1.2 Alignment of Approaches

The alignment of the approach legs plays an important role in the design of a roundabout. The alignment affects the amount of deflection (speed control) that is achieved, the ability to accommodate the design vehicle, and the visibility angles to adjacent legs. The optimal alignment is generally governed by the size and position of the roundabout relative to its approaches. Various options for approach alignment are summarized in Figure 19-6.

A common starting point in design is to center the roundabout so that the centerline of each leg passes through the center of the inscribed circle (radial alignment). This location typically allows the geometry of a single-lane roundabout to be adequately designed such that vehicles will maintain slow speeds through both the entries and the exits. The radial alignment also makes the central island more conspicuous to approaching drivers and minimizes roadway modification required upstream of the intersection.

Another frequently acceptable alternative is to offset the centerline of the approach to the left (i.e., the centerline passes to the left of the roundabout's center point). This alignment will typically increase the deflection achieved at the entry to improve speed control. However, engineers should recognize the inherent tradeoff of a larger radius (or tangential) exit that may provide less speed control for the downstream pedestrian crossing. Especially in urban environments, it is important to have drivers maintain sufficiently low vehicular speeds at the pedestrian crossing to reduce the risk for pedestrians. The fastest-path procedure provided in Section 19.7.1 identifies a methodology for estimating speeds for large radius (or tangential) exits where acceleration may govern the attainable speed.

Approach alignments that are offset to the right of the roundabout's center point typically do not achieve satisfactory results, primarily due to a lack of deflection and lack of speed control that result from this alignment. An offset-right alignment brings the approach in at a more tangential angle and reduces the opportunity to provide sufficient entry curvature. Vehicles will usually be able to enter the roundabout too fast, resulting in more loss-of-control crashes and higher crash rates between entering and circulating vehicles. However, an offset-right alignment alone should not be considered a fatal flaw in a design if speed requirements and other design considerations can be met.

Like signalized and stop-controlled intersections, the angle between approach legs is also an important design consideration. Although it is not necessary for opposing legs to align directly opposite one another (as it is for conventional intersections), it is generally preferable for the approaches to intersect at perpendicular or near-perpendicular intersection angles. If two approach legs intersect at an angle significantly greater than 90°, it will often result in excessive speeds for one or more right-turn movements. Alternatively, if two

Entry Alignment

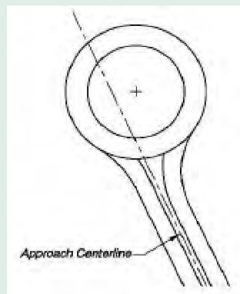
Question

Should the approach alignment run through the center of the inscribed circle? Or is it acceptable to offset the approach centerline to one side?

Design Principle

The alignment does not have to pass through the center of the roundabout; however, it has a primary effect on the entry/exit design. The optimal alignment allows for an entry design that provides adequate deflection and speed control while also providing appropriate view angles to drivers and balancing property impacts/costs.

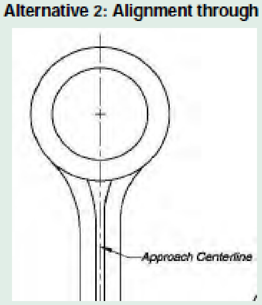
Alternative 1: Offset Alignment to the Left of Center



ADVANTAGES:
- Allows for increased deflection
- Beneficial for accommodating large trucks with small inscribed circle diameter—allows for larger entry radius while maintaining deflection and speed control
- May reduce impacts to right-side of roadway

TRADE-OFFS
- Increased exit radius or tangential exit reduces control of exit speeds and acceleration through crosswalk area
- May create greater impacts to the left side of the roadway

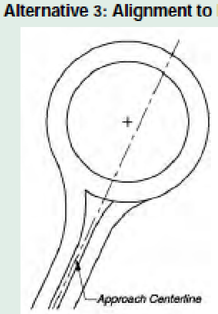
Alternative 2: Alignment through Center of Roundabout



ADVANTAGES:
- Reduces amount of alignment changes along the approach roadway to keep impacts more localized to intersection
- Allows for some exit curvature to encourage drivers to maintain slower speeds through the exit

TRADE-OFFS
- Increased exit radius reduces control of exit speeds/acceleration through crosswalk area
- May require a slightly larger inscribed circle diameter (compared to offset-left design) to provide the same level of speed control

Alternative 3: Alignment to Right of Center



ADVANTAGES:
- Could be used for large inscribed circle diameter roundabouts where speed control objectives can still be met
- Although not commonly used, this strategy may be appropriate in some instances (provided that speed objectives are met) to minimize impacts, improve view angles, etc.

TRADE-OFFS
- Often more difficult to achieve speed control objectives, particularly at small diameter roundabouts
- Increases the amount of exit curvature that must be negotiated

Figure 19-6 [NCHRP Report 672 Exhibit 6-10 (1)] Entry Alignment Alternatives

approach legs intersect at an angle significantly less than 90°, then the difficulty for large trucks to successfully navigate the turn is increased. Providing a large corner radius to accommodate trucks may result in a wide portion of circulatory roadway resulting in increased speeds and may also lead to reduced safety performance if the circulatory roadway width is mistakenly interpreted by drivers to be two lanes.

Designing the approaches at perpendicular or near-perpendicular angles generally results in relatively slow and consistent speeds for all movements. Highly skewed intersection angles can often require significantly larger inscribed circle diameters to achieve the speed objectives. Approaches that intersect at angles greater than approximately 105° can be realigned by introducing curvature in advance of the roundabout to produce a more perpendicular intersection.

Other possible geometric modifications include changes to the inscribed circle diameter or modifications to the shape of the central island to manage vehicle speeds. For roundabouts in low-speed urban environments, the alignment of the approaches may be less critical.

19.3.1.3 Design Vehicle

The design vehicle will dictate many of the roundabout's dimensions and the designer should consider the largest design vehicle to normally use that facility. Consult Chapter 2, Design Controls and Criteria, for more information regarding the appropriate design vehicle.

Because roundabouts are intentionally designed to slow traffic, narrow curb-to-curb widths and tight turning radii are typically used. However, if the widths and turning requirements are designed too tight, it can create difficulties for large vehicles. Large trucks and buses often dictate many of the roundabout's dimensions, particularly for single-lane roundabouts. Nearly all roundabouts feature truck aprons, which provides additional paved surface to accommodate the wide path of the trailer, but keeps the actual circulatory roadway width narrow enough to maintain speed control for smaller passenger cars.

19.4 SINGLE-LANE ROUNDABOUTS

This section presents general guidelines for the design of individual geometric elements at a single-lane roundabout. Many of these same principles also apply to the design of multilane roundabouts; however, there are some additional complexities to the design of multilane roundabouts that are described in detail in Section 19.5.

19.4.1 Splitter Islands

Splitter islands (also called separator islands or median islands) should be provided on all single-lane roundabouts. Their purpose is to provide refuge for pedestrians, assist in controlling speeds, guide traffic into the roundabout, physically separate entering and exiting traffic streams, and deterring wrong-way movements.

When performing the initial layout of a roundabout's design, a sufficiently sized splitter island envelope should be identified prior to designing the entry and exits of an approach. This will ensure that the design will eventually allow for a raised island that meets the minimum dimensions (offsets, tapers, length, widths).

The total length of the splitter island will vary based on terrain, access considerations, site-specific mainline and crossroad operational speeds, and the stepdown speeds to the final desired entry speed. However, the raised island should be at least 50 feet in length (100 feet is desirable) to provide sufficient protection for pedestrians and to alert approaching drivers to the geometry of the roundabout. On higher speed roadways, splitter island lengths of 150 feet or more are often beneficial. Additionally, the splitter island should extend beyond the end of the exit curve to prevent exiting traffic from accidentally crossing into the path of approaching traffic. The splitter island width should be a minimum of 6 feet at the crosswalk to adequately provide refuge for pedestrians. Figure 19-7 shows the minimum dimensions for a splitter island at a single-lane roundabout, including the location of the pedestrian crossing.

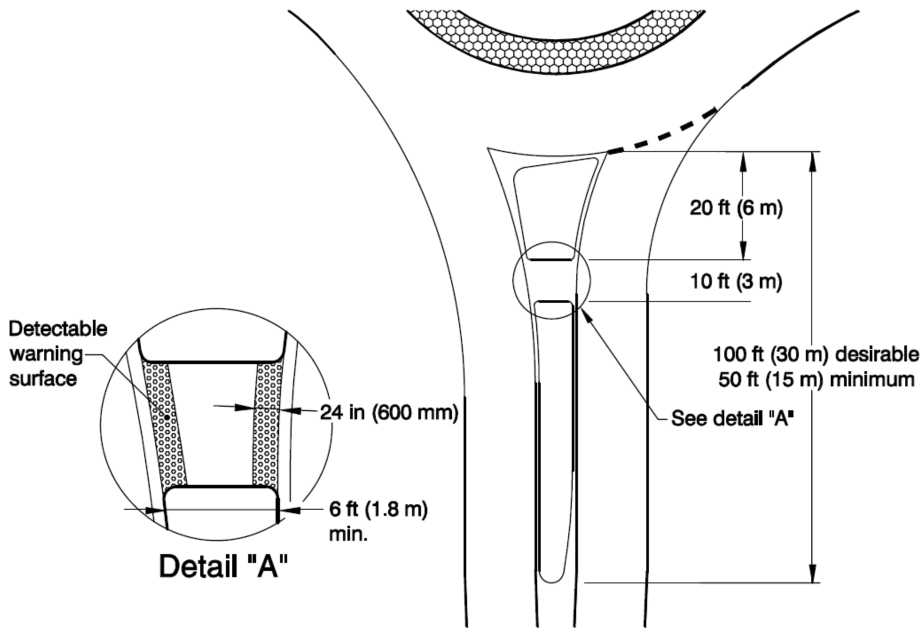


Figure 19-7 [NCHRP Report 672 Exhibit 6-12 (1)] Minimum Splitter Island Dimensions

While Figure 19-7 provides minimum dimensions for splitter islands, there are benefits to providing larger islands. An increase in the splitter island width results in greater separation between the entering and exiting traffic streams of the same leg and increases the time for approaching drivers to distinguish between exiting and circulating vehicles. This results in better gap acceptance and can help reduce confusion for entering motorists. A larger splitter island width also supports better pedestrian refuge.

Standard AASHTO guidelines for island design should be followed for the splitter island. This includes using larger nose radii at approach corners to maximize island visibility and offsetting curb lines at the approach ends to create a funneling effect. The funneling treatment also aids in reducing speeds as vehicles approach the roundabout. Figure 19-8 shows typical minimum splitter island nose radii and offset dimensions from the entry and exit traveled ways.

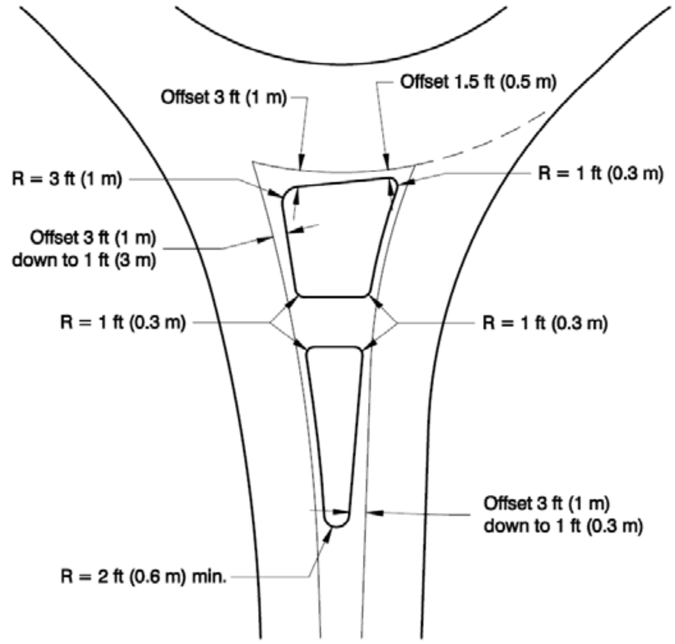


Figure 19-8 [NCHRP Report 672 Exhibit 6-13 (1)] Typical Minimum Splitter Island Nose Radii and Offsets

19.4.2 Entry Width

Typical entry widths for single-lane roundabout entrances range from 14 to 18 feet. These entries are often flared from upstream approach widths. However, values higher or lower than this range may be appropriate for site-specific design vehicle and speed requirements for critical vehicle paths. A 15-foot entry width is a common starting value for a single-lane roundabout. Care should be taken with entry widths greater than 18 feet or for those that exceed the width of the circulatory roadway, as drivers may mistakenly interpret the wide entry to be two lanes when there is only one receiving circulatory lane. Figure 19-9 shows a typical single-lane roundabout entry design.

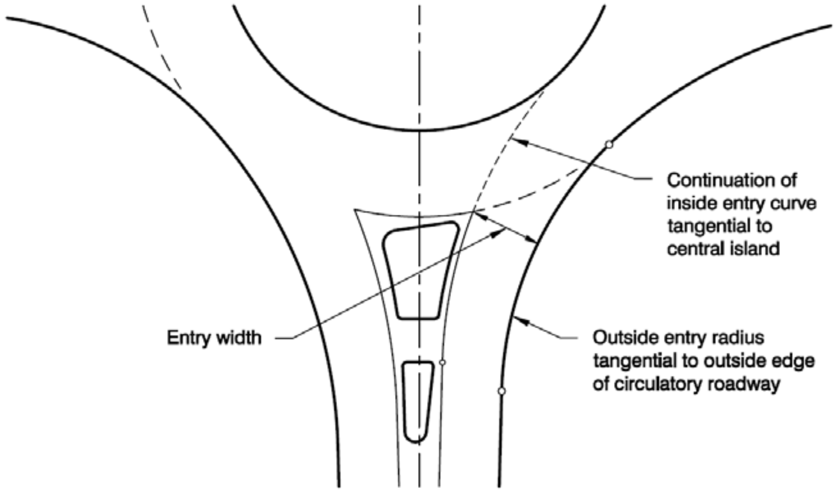


Figure 19-9 [NCHRP Report 672 Exhibit 6-14 (1)] Single-Lane Roundabout Entry Design

19.4.3 Circulatory Roadway Width

The circulating width should be at least as wide as the maximum entry width and up to 120% of the maximum entry width. For single-lane roundabouts, the circulatory roadway width usually remains constant throughout the roundabout. Typical circulatory roadway widths range from 16 to 20 feet for single-lane roundabouts. Care should be taken to avoid making the circulatory roadway width too wide within a single-lane roundabout because drivers may think that two vehicles are allowed to circulate side-by-side. Typically, the circulatory roadway width should typically be designed to accommodate the swept path of a bus design vehicle without use of the truck apron to avoid jostling bus passengers by running over the truck apron.

19.4.4 Central Island & Truck Apron

The central island of a roundabout is the raised, mainly non-traversable area surrounded by the circulatory roadway. It may also include a traversable truck apron. The island is typically landscaped for aesthetic reasons and to enhance driver recognition of the roundabout upon approach. Raised central islands for roundabouts are preferred over depressed central islands on the Colorado State Highway system.

Truck aprons should be designed such that they are traversable to trucks but discourage passenger vehicles from using them. Truck apron width is dictated by the swept path of the design vehicle using a CAD-based vehicle turning path simulation software (see Figure 19-10). Truck aprons should generally be 3 to 15 feet wide and have a cross slope of 1% to 2% away from the central island. To discourage use by passenger vehicles, the outer edge of the apron should be raised approximately 2 to 3 inches above the circulatory roadway surface. The apron should be constructed of a different material than the pavement to differentiate it from the circulatory roadway.

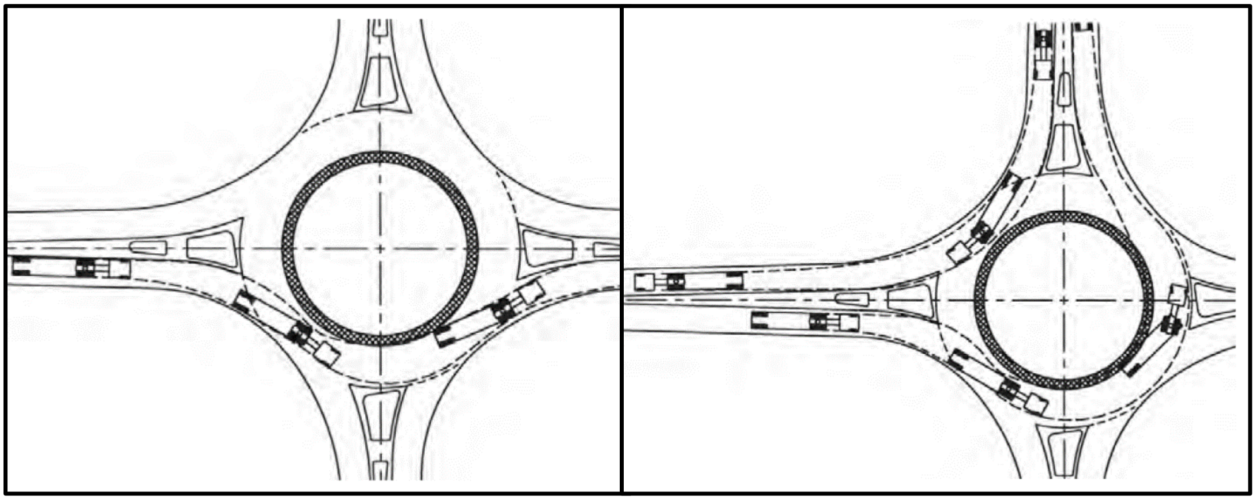


Figure 19-10 [NCHRP Report 672 Exhibits 6-17 & 6-18 (1)] Typical Swept Path of a Large Design Vehicle through a Single-Lane Roundabout

19.4.5 Entry Design

At single-lane roundabouts, a single-entry curb radius is typically adequate. For approaches on higher speed roadways, the use of compound curves may improve guidance by lengthening the entry arc.

The entry curb radius, in conjunction with the entry width, the circulatory roadway width, and the central island geometry, controls the amount of deflection imposed on a vehicle's entry path. Excessively large entry curb radii have a higher potential to produce faster entry speeds than desired.

Entry radii at urban single-lane roundabouts typically range from 50 to 100 feet. A common starting point is an entry radius in the range of 60 to 90 feet; however, a larger or smaller radius may be needed to accommodate large vehicles or serve small diameter roundabouts, respectively. Larger radii may be used, but it is important that the radii not be so large as to result in excessive entry speeds.

The entry geometry should provide adequate horizontal curvature to channelize drivers into the circulatory roadway to the right of the central island. It is also often desirable for the splitter island to have enough curvature to block a direct path to the central island for approaching vehicles. To achieve the proper amount of deflection for each approach to a roundabout, an entry angle usually between 20° and 40° is desirable. Not only does the entry angle aid in the slowing the vehicle entry speed, it also helps so vehicles don't hit broadside in the event of a collision. Figure 19-11 depicts the roundabout entry angle.

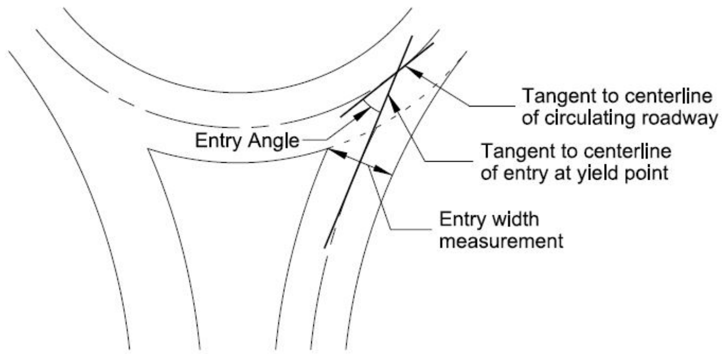


Figure 19-11 Roundabout Entry Angle

19.4.6 Exit Design

The exit curb radii are usually larger than the entry curb radii in order to minimize the likelihood of congestion and crashes at the exits. This, however, is balanced by the need to maintain slow speeds through the pedestrian crossing on exit. The exit design is also influenced by the design environment (urban versus rural), pedestrian demand, the design vehicle, and physical site constraints.

The exit curb is commonly designed to be curvilinearly tangential to the outside edge of the circulatory roadway. Likewise, the projection of the inside (left) edge of the exit roadway is commonly curvilinearly tangential to the central island. Generally, exit curb radii should be no less than 50 feet, with values of 100 to 200 feet being more common. Figure 19-12 shows a typical exit layout for a single-lane roundabout.

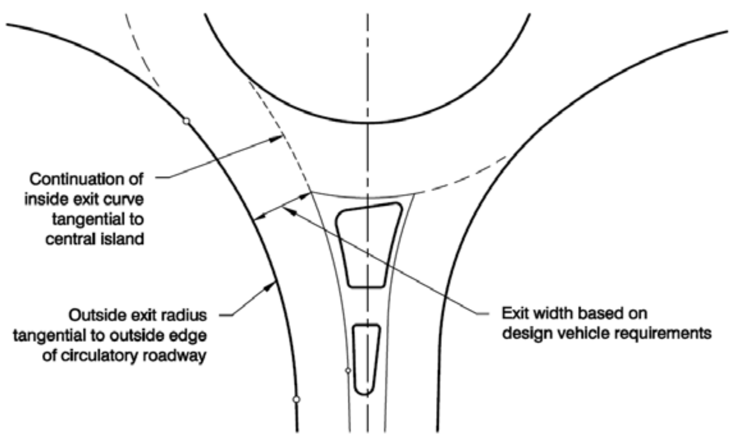


Figure 19-12 [NCHRP Report 672 Exhibit 6-15 (1)] Single-Lane Roundabout Curvilinear Exit Design

For designs using an offset-left approach alignment, the exit design may require much larger radii, ranging from 300 to 800 feet or greater. Larger exit radii may also be desirable in areas with high truck volumes to provide ease of navigation for trucks and reduce the potential for trailers to track over the outside curb. These radii may provide acceptable speed through the pedestrian crossing area given that the acceleration characteristics of the vehicles will result in a practical limit to the speeds that can be achieved on the exit. Figure 19-13 depicts the larger radius exit design of a single-lane roundabout.

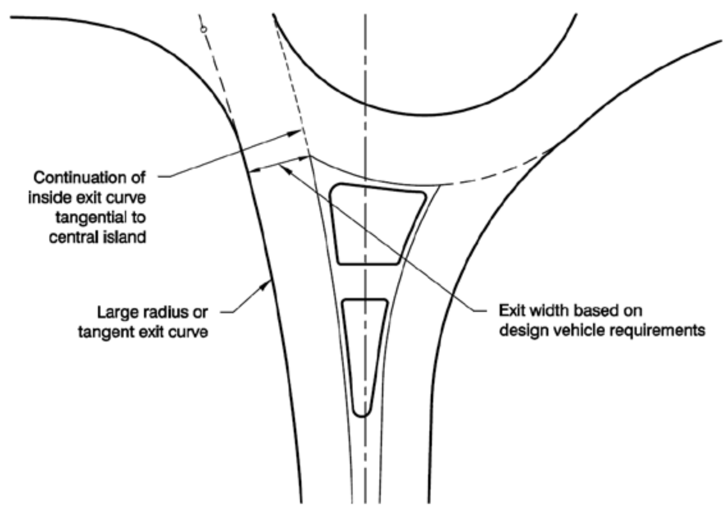


Figure 19-13 [NCHRP Report 672 Exhibit 6-16 (1)] Single-Lane Roundabout Larger Radius Exit Design

19.4.7 Right-Turn Bypass Lanes

Right-turn bypass lanes are a proven way to increase the "life" of a single-lane roundabout by removing traffic that would otherwise enter the roundabout and reduce the available capacity to other movements. Extending the life of a single-lane roundabout is desirable given the stronger safety performance in comparison to multilane roundabouts due to the smaller size and slower speeds that are achieved. To determine if a right-turn bypass lane should be used, the appropriate capacity and delay calculations should be performed.

A right-turn bypass lane should be implemented only where needed. In urban areas with heavy bicycle and pedestrian activity, a right-turn bypass lane should be used with caution. The entries and exits of the bypass lane can increase conflicts with bicyclists and with merging maneuvers on the downstream leg. The generally higher speeds of bypass lanes and the lower expectation of drivers to stop may increase the risk of collisions with pedestrians. They also introduce additional complexity for pedestrians with visual impairments who are attempting to navigate the intersection. However, in locations with minimal pedestrian and bicycle activity, or where bicycle and pedestrian concerns can be addressed through design solutions, right-turn bypass lanes can be used to improve capacity when heavy right-turning traffic exists. Figure 19-14 shows a sample layout of a right-turn bypass lane for a single-lane roundabout.

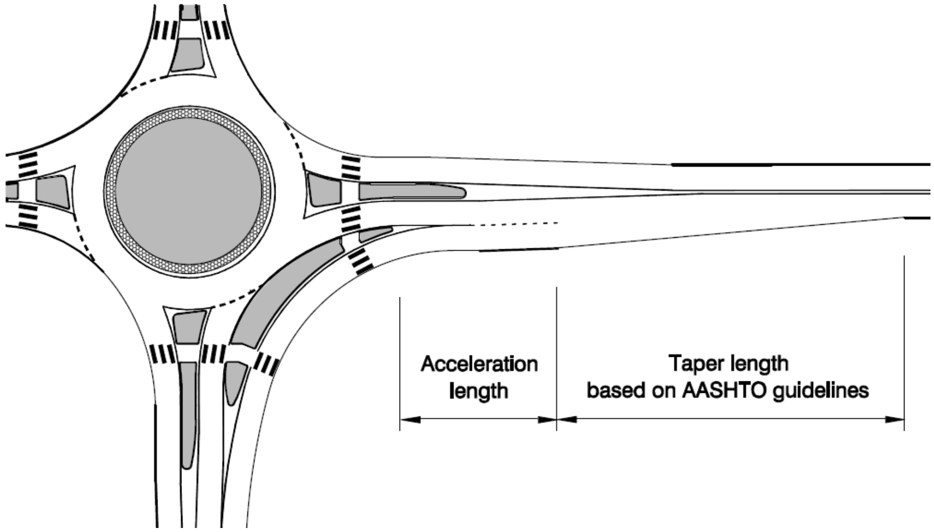


Figure 19-14 [NCHRP Report 672 Exhibit 6-72 (1)] Sample Layout of Right-Turn Bypass Lane with Acceleration Lane

19.5 MULTILANE ROUNDABOUTS

The principles and design process described previously for single-lane roundabouts also apply to multilane roundabouts but in a more complex way. Because multiple traffic streams may enter, circulate through, and exit the roundabout side-by-side, the engineer should consider how these traffic streams interact with each other. The geometry of the roundabout should provide adequate alignment and establish appropriate lane configurations for vehicles in adjacent entry lanes to be able to negotiate the roundabout geometry without competing for the same space.

The number of lanes within the circulatory roadway may vary depending upon the number of entering and exiting lanes. The important principle is that the design requires continuity between the entering, circulating, and exiting lanes such that lane changes are not needed to navigate the roundabout. The driver should be able to select the appropriate lane upstream of the entry and stay within that lane through the roundabout to the intended exit without any lane changes.

The number of lanes provided at the roundabout should be the minimum needed for the existing and anticipated demand as determined by the operational analysis. The engineer is discouraged from providing additional lanes that are not needed for capacity purposes as these additional lanes can reduce the safety effectiveness at the intersection. If additional lanes are needed for future conditions, a phased design approach should be considered that would allow for future expansion.

19.5.1 Entry Width

A typical entry width for a multilane roundabout ranges from 24 to 30 feet for a two-lane entry and from 36 to 45 feet for a three-lane entry. Typical widths for individual lanes at entry range from 12 to 15 feet. The entry width should be primarily determined based upon the number of lanes identified in the operational analysis combined with the turning requirements for the design vehicle. Excessive entry width may not produce capacity benefits if the entry width cannot be fully used by traffic.

At locations where any of the intersection approach legs is a 2-lane roadway, but a multilane roundabout capacity is required to meet the operational needs, there are generally two options for developing the second roundabout entry lane:

1. Adding a full lane upstream of the roundabout and maintaining parallel lanes through the entry geometry (Figure 19-15)
2. Widening the approach by gradually flaring through the entry geometry (Figure 19-16).

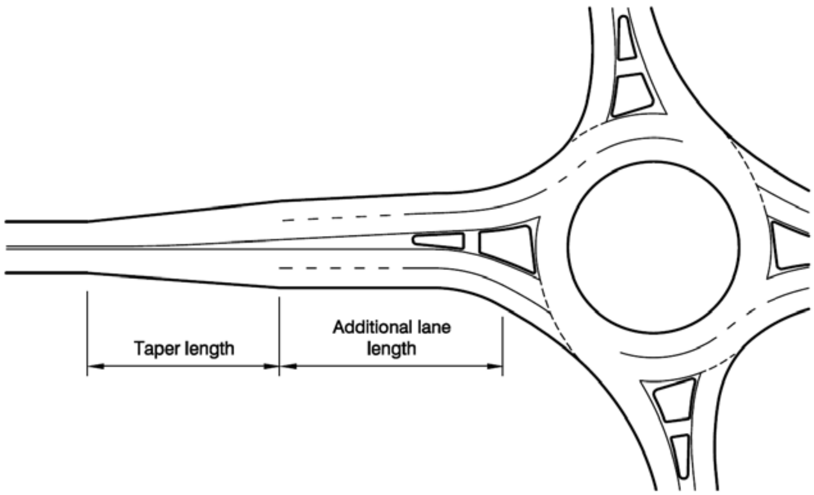


Figure 19-15 [NCHRP Report 672 Exhibit 6-24 (1)] Approach Widening by Adding a Full Lane

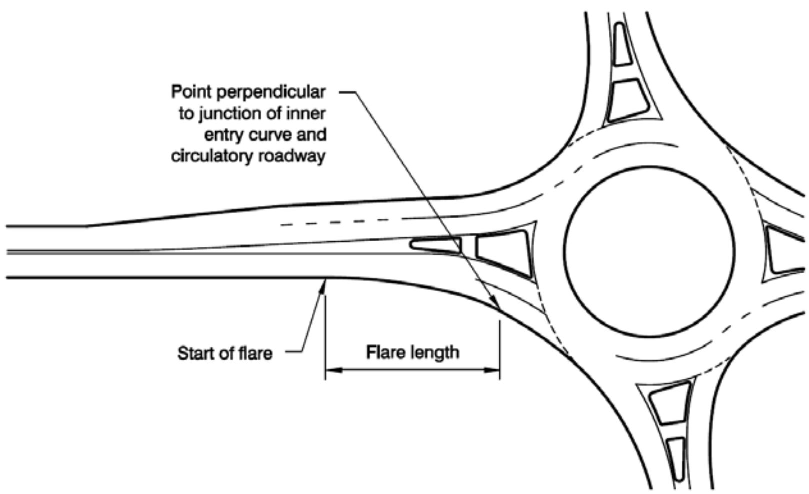


Figure 19-16 [NCHRP Report 672 Exhibit 6-25 (1)] Approach Widening by Entry Flaring

19.5.2 Circulatory Roadway Widths

The circulatory roadway width for multilane roundabouts is usually governed by the design criteria relating to the types of vehicles that may need to be accommodated adjacent to one another. If the entering traffic is predominantly passenger cars and single-unit trucks (AASHTO P and SU design vehicles, respectively) and semi-trailer traffic is infrequent, it may be appropriate to design the width for two passenger vehicles or a passenger car and a single-unit truck side-by-side. If semi-trailer traffic is relatively frequent (greater than 10%), it may be necessary to provide sufficient width for the simultaneous passage of a semi-trailer in combination with a P or SU vehicle.

Multilane circulatory roadway lane widths typically range from 14 to 16 feet. Use of these values results in a total circulating width of 28 to 32 feet for a two-lane circulatory roadway and 42 to 48 feet total width for a three-lane circulatory roadway.

A constant width is not required throughout the entire circulatory roadway. It is desirable to provide only the minimum width necessary to serve the required lane configurations within that specific portion of the roundabout. A common combination is two entering and exiting lanes along the major roadway, but only single entering and exiting lanes on the minor street (Figure 19-17).

In some instances, the circulatory roadway width may need to be wider than the corresponding entrance that is feeding that portion of the roundabout. For example, in situations where two consecutive entries require exclusive left turns, a portion of the circulatory roadway will need to contain an extra lane and spiral markings to enable all vehicles to reach their intended exits without being trapped or changing lanes (Figure 19-18).

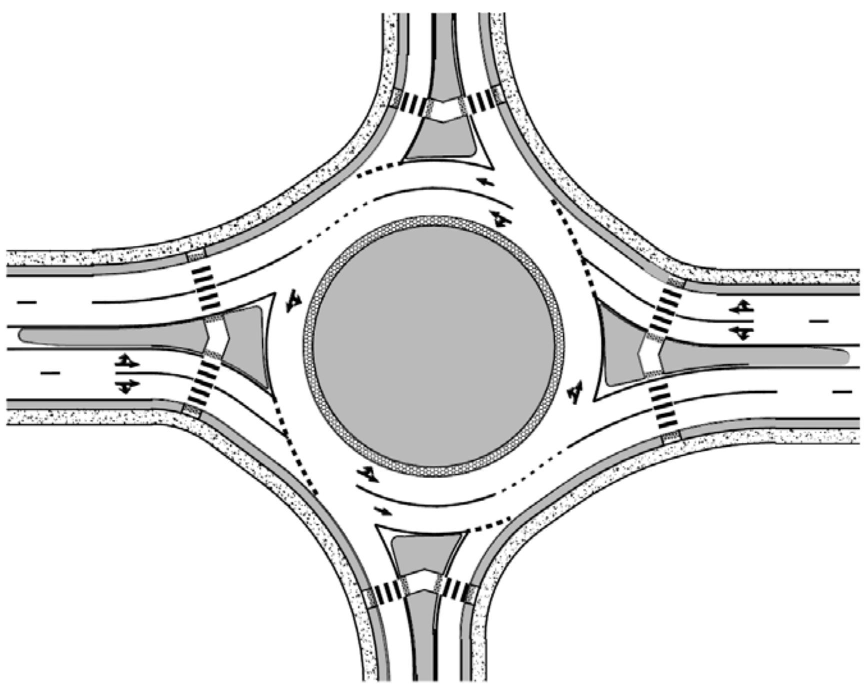


Figure 19-17 [NCHRP Report 672 Exhibit 6-26 (1)] Multilane Major Street with Single-Lane Minor Street

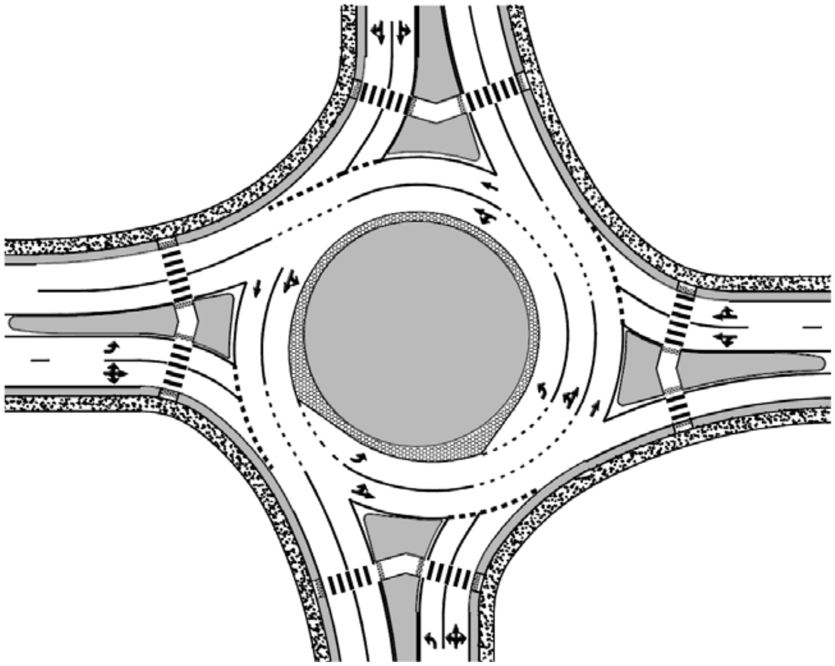


Figure 19-18 [NCHRP Report 672 Exhibit 6-27 (1)] Two-Lane Roundabout with Consecutive Double-Lefts

19.5.3 Entry Geometry and Approach Alignment

Entry radii for multilane roundabouts should typically exceed 65 feet to encourage adequate natural paths and avoid sideswipe collisions on entry. Engineers should avoid the use of overly tight geometrics in order to achieve the fastest-path objectives. Overly small (less than 45 feet) entry radii can result in conflicts between adjacent traffic streams, which may result in poor lane use and reduced capacity. Similarly, the R_1 fastest-path radius should also not be excessively small. If R_1 is too small, vehicle path overlap may result, reducing the operational efficiency and increasing potential for crashes. Values for R_1 in the range of 175 to 275 feet are generally preferable. This results in a design speed of 25 to 30 mph. Refer to Section 19.7.1 for more discussion on the fastest path guidelines.

One possible technique to promote good path alignment is shown in Figure 19-19 using a compound curve or tangent along the outside curb. The design consists of an initial small-radius entry curve set back from the edge of the circulatory road-way. A short section of a large-radius curve or tangent is provided between the entry curve and the circulatory roadway to align vehicles into the proper circulatory lane at the entrance line.

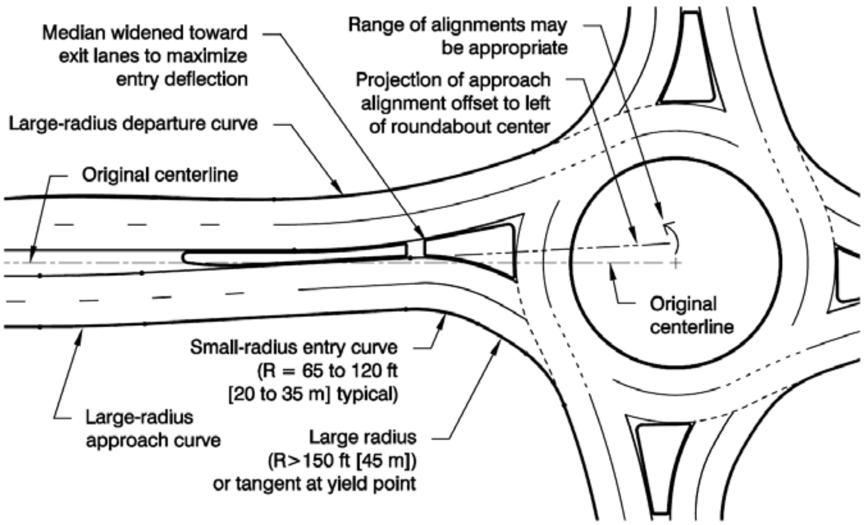


Figure 19-19 [NCHRP Report 672 Exhibit 6-30 (1)] Example Minor Approach Offset to Increase Entry Deflection

For the method illustrated in Figure 19-19, entry curve radii commonly range from approximately 65 to 120 feet and are set back at least 20 feet from the edge of the circulatory roadway. A tangent or large-radius (greater than 150 feet) curve is then fitted between the entry curve and the outside edge of the circulatory roadway.

19.5.3.1 Entry Geometry and Design Vehicle Considerations

Where the design dictates the need to accommodate large design vehicles within their own lane, there are a number of design considerations. A larger inscribed circle diameter and entry/exit radii may be required to maintain speed control and accommodate the design vehicle. A common technique that can be used is to provide gore striping between the two entry lanes to help center the vehicles within the lane and allow a cushion for off-tracking by the design vehicle. This technique is illustrated in Figure 19-20.

Another technique for accommodating the design vehicle within the circulatory roadway is to use a wider lane width for the outside lane and a narrower lane width for the inside lane. This could provide an extra buffer of circulating width for trucks in the outside lane. Large trucks in the inside lane would use the truck apron to accommodate any off-tracking. Eliminating all overlap for the outside lane may not always be desirable or feasible, as this may dictate a much larger inscribed circle diameter than desired for overall safety performance for all vehicle types and the context.

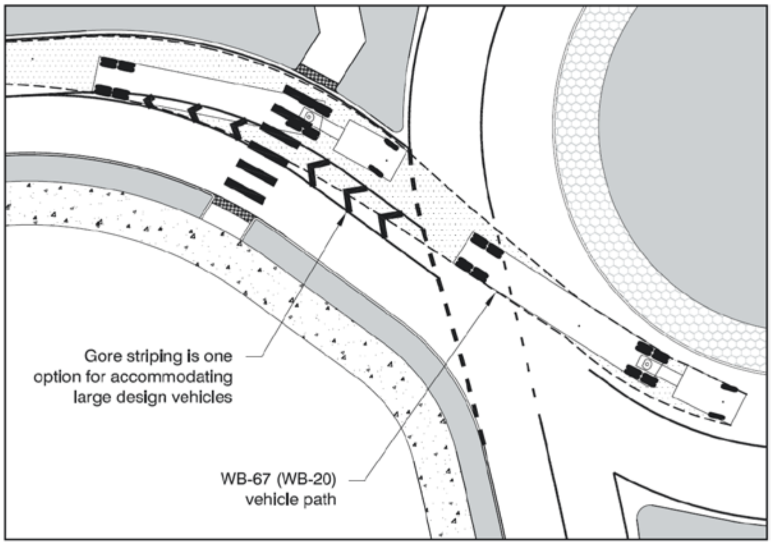


Figure 19-20 [NCHRP Report 672 Exhibit 6-37 (1)] Truck Path with Gore Striping at Entry

19.5.4 Path Overlap

In a multilane roundabout, the designer should avoid a design that aligns an entering vehicle at the incorrect lane in the circulatory roadway which will create path overlap (see Figure 19-21). As a vehicle enters the circulating roadway it should be headed directly toward its respective lane within the circulating roadway. Figure 19-22 illustrates the design vehicle path alignment of a multilane roundabout.

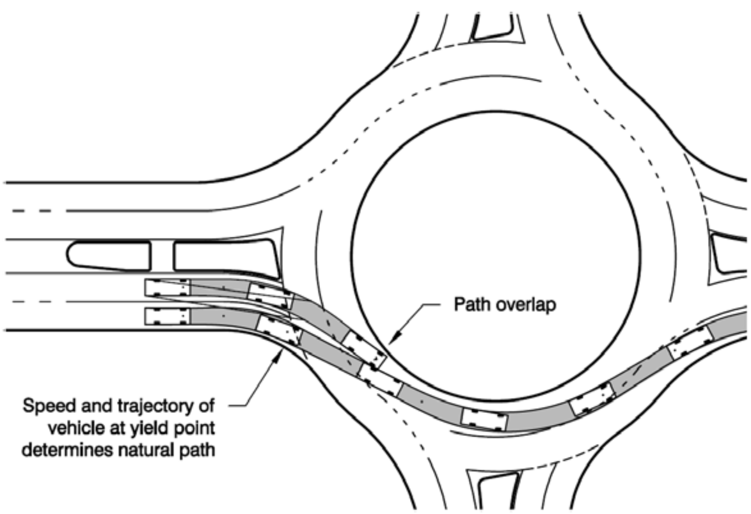


Figure 19-21 [NCHRP Report 672 Exhibit 6-28 (1)] Entry Vehicle Path Overlap

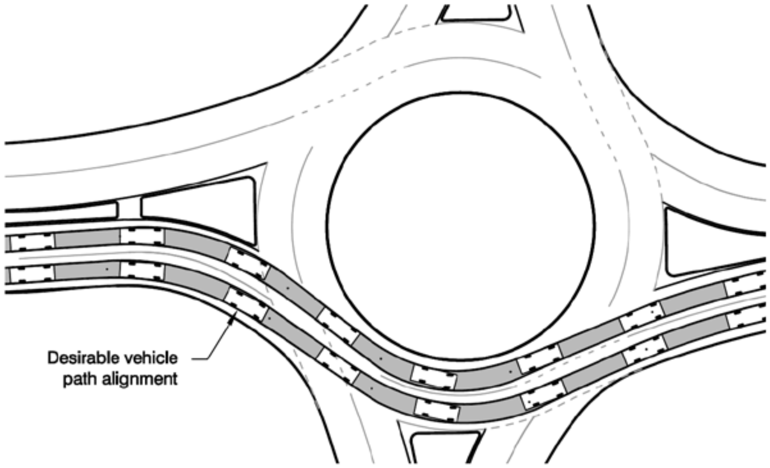


Figure 19-22 [NCHRP Report 672 Exhibit 6-29 (1)] Desirable Vehicle Path Alignment

19.5.5 Exit Curves

Conflicts can occur between exiting and circulating vehicles if appropriate lane assignments are not provided. Inadequate horizontal design of the exits can also result in exit vehicle path overlap, similar to that occurring at entries. The radii of exit curves are commonly larger than those used at the entry because of other factors (entry alignment, diameter, etc.); larger exit curve radii are also typically used to promote good vehicle path alignment. However, the design should be balanced to maintain low speeds at the pedestrian crossing at the exit.

To promote good path alignment at the exit, the exit radius at a multilane roundabout should not be too small. At single-lane roundabouts, it is acceptable to use a minimal exit radius in order to control exit speeds and maximize pedestrian safety. However, if the exit radius on a multilane exit is too small, traffic on the inside of the circulatory roadway will tend to exit into the outside exit lane on a more comfortable turning radius.

Problems can also occur when the design allows for too much separation between entries and subsequent exits. Large separations between legs causes entering vehicles to join next to circulating traffic that may be intending to exit at the next leg, rather than crossing the path of the exiting vehicles. This can create conflicts at the exit point between exiting and circulating vehicles, as shown in Figure 19-23.

While it would be possible to provide a low-cost solution by modifying the lane arrangements using a combination of striping and other physical modifications, a better solution would be to realign the approach legs to have the paths of entering vehicles cross the paths of the circulating traffic (rather than merging) to eliminate the conflict as shown in Figure 19-24.

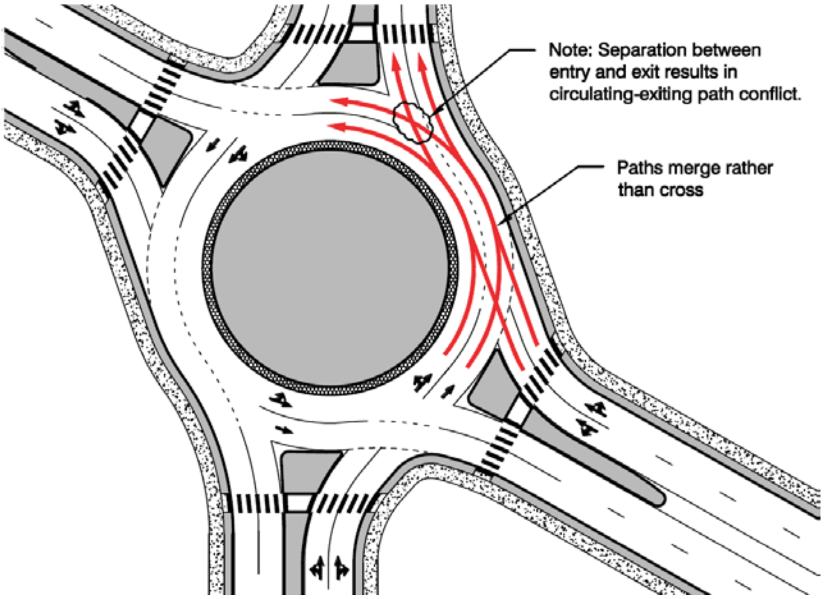


Figure 19-23 [NCHRP Report 672 Exhibit 6-33 (1)] Exit-Circulating Conflict Caused by Large Separation between Legs

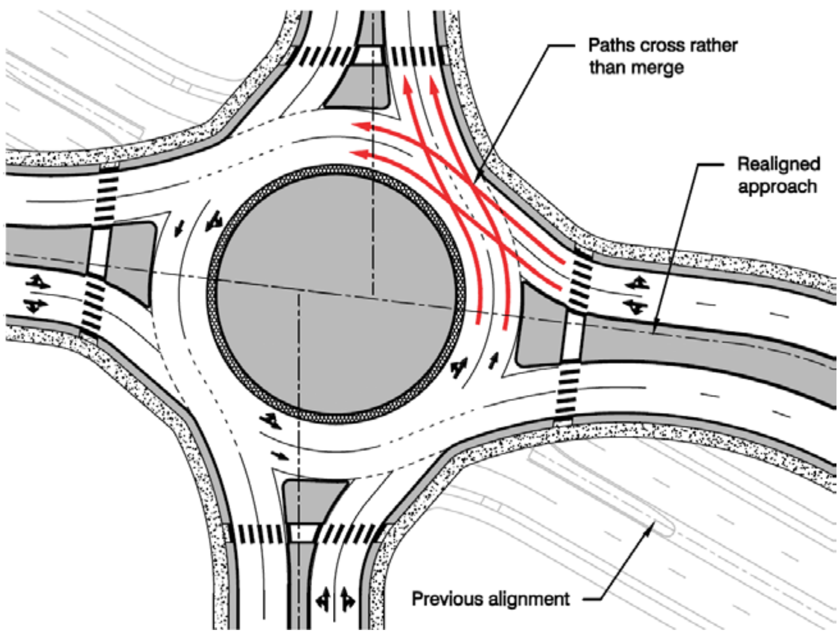


Figure 19-24 [NCHRP Report 672 Exhibit 6-35 (1)] Realignment to Resolve Exit-Circulating Conflicts

19.6 MINI-ROUNDABOUTS

As discussed in Section 19.1.1, Mini-roundabouts are small single-lane roundabouts with a fully traversable central island that are most commonly used in low-speed urban environments with average operating speeds of 30 mph or less. Given that the central island of a mini-roundabout is fully traversable, the overall design should provide channelization that naturally guides drivers to the intended path. Sub-optimum designs may result in drivers turning left in front of the central island (or driving over the top of it), improperly yielding, or traveling at excess speeds through the intersection.

Mini-roundabouts should be made as large as possible within the intersection constraints. However, a mini-roundabout inscribed circle diameter should generally not exceed 90 feet. Above 90 feet, the inscribed circle diameter is typically large enough to accommodate the design vehicles navigating around a raised central island. A raised central island provides physical channelization to control vehicle speeds, and therefore a single-lane design is preferred where a diameter greater than 90 feet can be provided.

As with single-lane and multilane roundabouts, it is desirable to accommodate buses within the circulatory roadway to avoid jostling passengers by running over a traversable central island. However, for very small inscribed circle diameters, the bus turning radius is typically too large to navigate around the central island, thus requiring buses to travel over it. For mini-roundabouts with larger inscribed circle diameters, it may be possible to accommodate the swept path of a bus vehicle within the circulatory roadway. The potential trade-off to designing for a bus instead of a passenger car is that the design may result in a wider circulatory roadway and smaller central island.

Composed of asphalt concrete, Portland cement concrete, or other paving material, the central island should be domed using 5% to 6% of cross slope, with a maximum height of 5 inches. Although fully traversable and relatively small, it is essential that the central island be clear and conspicuous. Islands with a mountable curb should be designed in a similar manner to truck aprons on normal roundabouts.

19.6.1 Splitter Islands

As with larger roundabouts, splitter islands are generally used at mini-roundabouts to align vehicles, encourage deflection and proper circulation, and provide pedestrian refuge. Splitter islands are raised, traversable, or flush depending on the size of the island and whether trucks will need to track over the top of the splitter island to navigate the intersection. In general, raised islands are used where possible, and flush islands are generally discouraged. The following are general guidelines for the types of splitter islands under various site conditions

Consider a raised island if:

- All design vehicles can navigate the roundabout without tracking over the splitter island area,
- Sufficient space is available to provide an island with a minimum area of 50 ft^2, and/or
- Pedestrians are present at the intersection with regular frequency.

Consider a traversable island if:

- Some design vehicles must travel over the splitter island area and truck volumes are minor, and
- Sufficient space is available to provide an island with a minimum area of 50 ft^2.

Consider a flush (painted) island if:

- Vehicles are expected to travel over the splitter island area with relative frequency to navigate the intersection,
- An island with a minimum area of 50 ft^2 cannot be achieved, and
- Intersection has slow vehicle speeds.

19.6.2 Pedestrian and Bicycle Treatments

At conventional intersections, pedestrian ramps and crosswalks are typically located near the curb returns at the corners of the intersection. When converting to a mini-roundabout, these corner pedestrian-crossing locations may require relocation. The crosswalk is recommended to be located 20 feet upstream of the entrance line to accommodate one vehicle stopped between the crosswalk and the entrance line. Where a minimum splitter island width of 6 feet is available on the approach, a pedestrian refuge should be provided within the splitter island.

Bicyclists are encouraged to navigate through a mini-roundabout like other vehicles. Where bicycle lanes are provided on the approaches to a mini-roundabout, they should be terminated to alert motorists and bicyclists of the need for bicyclists to merge. Bike lanes should be terminated at least 100 feet upstream of the entrance line.

19.6.3 Vertical Design

Mini-roundabouts should be designed to be outward draining to place the central island at the highest point of the intersection for maximum visibility.

19.7 PERFORMANCE CHECKS

Performance checks are a vital part of the roundabout design process in order to help an engineer determine whether the design meets its performance objectives. The following are the critical performance checks that need to be performed prior to finalizing any roundabout design:

- Fastest Path
- Path Alignment
- Sight Distance
- Angles of Visibility

19.7.1 Fastest Path

The fastest path allowed by the roundabout geometry determines the negotiation speed for that particular movement into, through, and exiting the roundabout. It is the smoothest, flattest path possible for a single vehicle, in the absence of other traffic and ignoring all lane markings. The fastest path does not represent expected vehicle speeds, but rather theoretical attainable entry speeds for design purposes.

Maximum entering design speeds based on a theoretical fastest path of 20 to 25 mph are recommended at single-lane roundabouts. At multilane roundabouts, maximum entering design speeds of 25 to 30 mph are recommended. These speeds are influenced by a variety of factors, including the geometry of the roundabout and the operating speeds of the approaching roadways. As a result, speed management is often a combination of managing speeds at the roundabout itself and managing speeds on the approaching roadways.

There are five critical path radii that must be checked for each roundabout approach (Figure 19-25) as follows:

- **R_1** - the entry path radius, is the minimum radius on the fastest through path prior to the entrance line.
- **R_2** - the circulating path radius, is the minimum radius on the fastest through path around the central island.
- **R_3** - the exit path radius, is the minimum radius on the fastest through path into the exit.
- **R_4** - the left-turn path radius, is the minimum radius on the path of the conflicting left-turn movement.
- **R_5** - the right-turn path radius, is the minimum radius on the fastest path of a right-turning vehicle.

It is important to note that these vehicular path radii are not the same as the curb radii. The R_1 through R_5 radii measured in this procedure represent the vehicle centerline in its path through the roundabout.

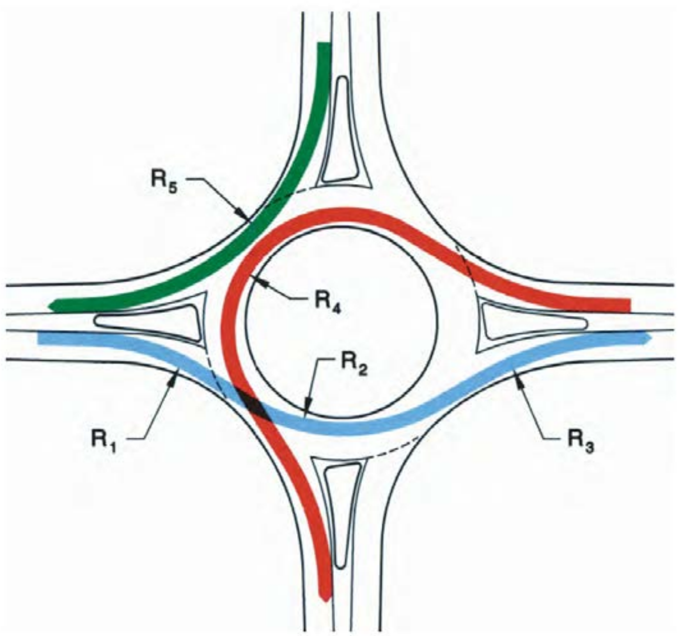


Figure 19-25 [NCHRP Report 672 Exhibit 6-46 (1)] Fastest Path Radii

Once a conceptual roundabout design is complete, the engineer should draw out the fastest path alignment to determine the speed of the roundabout. The design speed of the roundabout is determined from the smallest radius along the fastest allowable path. The smallest radius usually occurs on the circulatory roadway as the vehicle curves to the left around the central island.

A vehicle is assumed to be 6 feet wide and maintain a minimum clearance of 2 feet from a roadway centerline or concrete curb and flush with a painted edge line. Thus, the centerline of the vehicle path is drawn with the following distances:

- 5 feet from the face (flowline) of a concrete curb
- 5 feet from a roadway centerline

- 3 feet from a painted edge line

Figure 19-26 illustrates the construction of the fastest vehicle path alignment at a multilane roundabout.

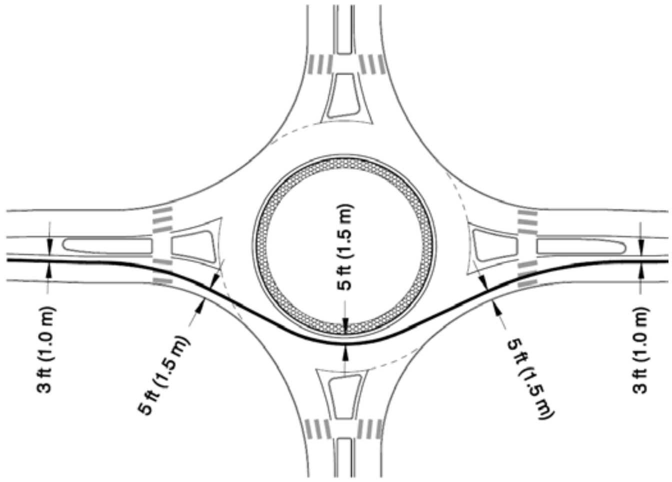


Figure 19-26 [NCHRP Report 672 Exhibit 6-48 (1)] Fastest Path Radii

The relationship between travel speed and horizontal curvature is documented in the *PGDHS* **(2).** Both superelevation and the side friction factor affect the speed of a vehicle. Side friction varies with vehicle speed and can be determined in accordance with AASHTO guidelines. The most common superelevation values encountered are +0.02 and −0.02, corresponding to 2% cross slope. Figure 19-27 depicts the speed-to-radius relationship in a graphical format.

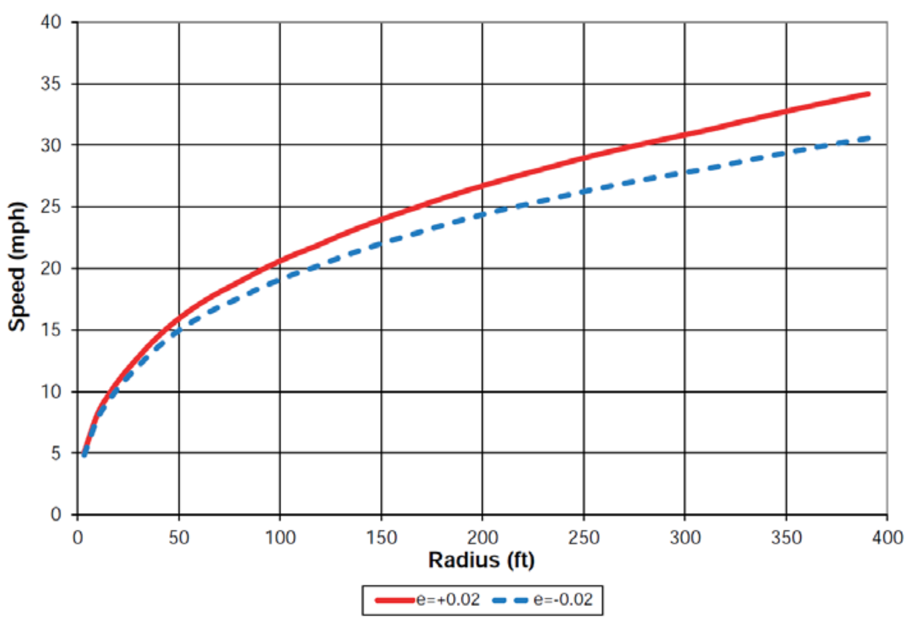


Figure 19-27 [NCHRP Report 672 Exhibit 6-52 (1)] Speed-to-Radius Relationship

The speed–radius relationship given above generally provides a reasonable prediction for the left-turn and through movement circulating speeds. However, this method does not consider the effects of deceleration and acceleration and therefore may overpredict entry and exit speeds in cases where the path radius is large

Consistency between the speeds of various movements within the intersection can help to minimize the crash rate between conflicting traffic streams. Relative speeds between conflicting traffic streams and between consecutive geometric elements should be minimized such that the maximum speed differential between movements should be no more than approximately 10 to 15 mph. These values are typically achieved by providing a low absolute maximum speed for the fastest entering movements. As with other design elements, speed consistency should be balanced with other objectives in establishing a design.

The desirable maximum R_1 radius is 150 feet for single-lane roundabouts and 250 feet for multilane roundabouts. Generally, for urban roundabouts with pedestrian accommodations a lower speed entry is desirable. Rural roundabouts typically allow slightly higher entry speed than urban roundabouts. The R_1 and R_2 should be used to control exit speed. Typically, the speed relationships between R_1, R_2, and R_3 as well as between R_1 and R_4 are of primary interest. Along the through path, the desired relationship is $R_1 > R_2 < R_3$, where R_1 is also less than R_3. Similarly, the relationship along the left-turning path is $R_1 > R_4$. For most designs, the $R_1 – R_4$ relationship will be the most restrictive for speed differential at each entry. However, the $R_1 – R_2 – R_3$ relationship should also be reviewed, particularly to ensure the exit speed is not overly restrictive. Design criteria in past years advocated relatively tight exit radii to minimize exit speed, however, recent best practice suggests a more relaxed exit radius for improved drivability.

19.7.2 Path Alignment (Natural Path) Considerations

As discussed in Section 19.7.1, the fastest path through the roundabout is drawn to ensure a safe design speed is achieved. In addition to evaluating the fastest path, at multilane roundabouts the engineer should also consider the natural vehicle paths. These are the paths approaching vehicles will naturally take through the roundabout geometry, assuming there is traffic in all approach lanes.

The key consideration in drawing the natural path is to remember that drivers cannot change the direction or speed of their vehicle instantaneously. This means that the natural path does not have sudden changes in curvature; it has transitions between tangents and curves and between consecutive reversing curves. Secondly, it means that consecutive curves should be of similar radius. If a second curve has a significantly smaller radius than the first curve, the driver will be traveling too fast to negotiate the turn and may not be able stay within the lane. If the radius of one curve is drawn significantly smaller than the radius of the previous curve, the path should be adjusted. As a rule of thumb, the design should provide at least one car length of large radius or tangent to adequately align vehicles into the correct lane within the circulatory roadway. Figure 19-28 illustrates a sample sketch of the natural path through a multilane roundabout.

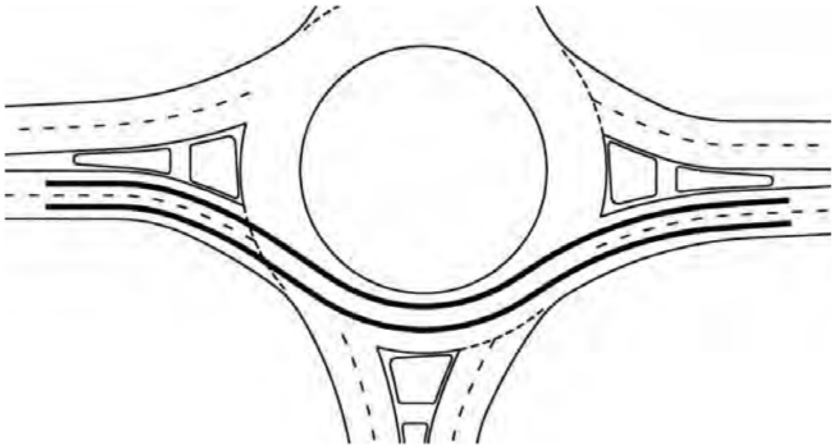


Figure 19-28 [NCHRP Report 672 Exhibit 6-53 (1)] Natural Vehicle Path Sketched through Roundabout

19.7.3 Sight Distance

The roundabout design should be checked to ensure adequate sight distance is achieved. The two most relevant aspects of sight distance for roundabouts are stopping sight distance and intersection sight distance. Stopping sight distance and intersection sight distance should be measured using an assumed height of the driver's eye of 3.5 feet and an assumed object height of 2 feet.

19.7.3.1 Stopping Sight Distance

Stopping sight distance should be provided at every point within a roundabout. *NCHRP Report 400: Determination of Stopping Sight Distance* recommends the formula given in Equation 19-1 for determining stopping sight distance.

$$\boldsymbol{d = (1.468)(t)(V) + 1.087\, V^2/a} \qquad [19\text{-}1]$$

where,

d = stopping sight distance, ft;

t = perception–brake reaction time, assumed to be 2.5 s;

V = initial speed, mph; and

a = driver deceleration, assumed to be 11.2 ft/s^2

Table 19-3 gives stopping sight distances computed from the above equations.

Speed (km/h)	Computed Distance* (ft)
10	46.4
15	77
20	112.4
25	152.7
30	197.8
35	247.8
40	302.7
45	362.5
50	427.2
55	496.7
* Assumes 2.5 s perception-breaking time, 11.2 ft/s^2 driver deceleration	

Table 19-3 [NCHRP Report 672 Exhibit 6-53 (1)] Stopping Sight Distance

At roundabouts, a minimum of three critical types of locations should be checked for stopping sight distance:

3. Approach sight distance (Figure 19-29),
4. Sight distance on circulatory roadway (Figure 19-30), and
5. Sight distance to crosswalk on exit (Figure 19-31).

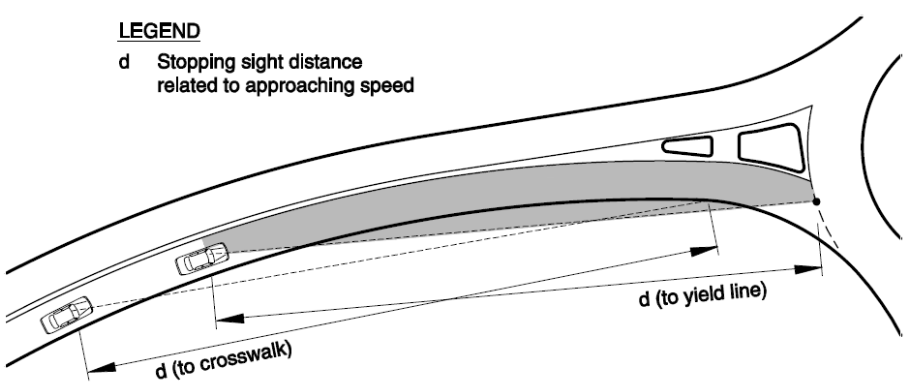


Figure 19-29 [NCHRP Report 672 Exhibit 6-55 (1)] Stopping Sight Distance on Approach

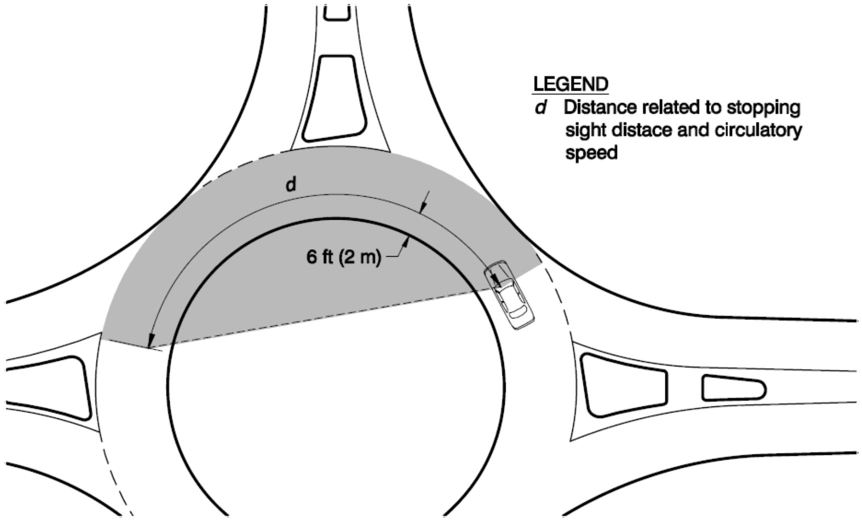


Figure 19-30 [NCHRP Report 672 Exhibit 6-56 (1)] Stopping Sight Distance on Circulatory Roadway

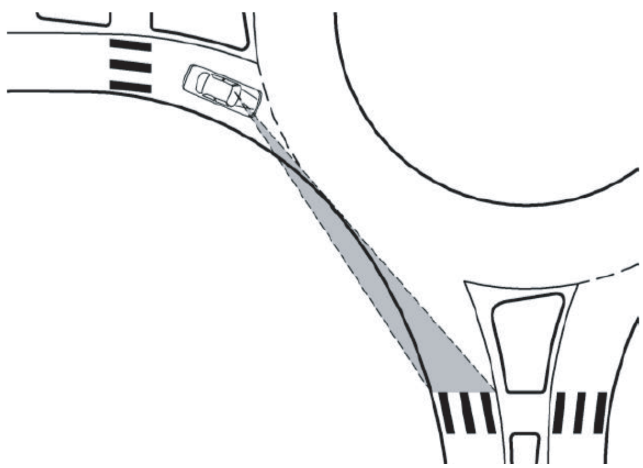


Figure 19-31 [NCHRP Report 672 Exhibit 6-57 (1)] Sight Distance to Crosswalk on Exit

19.7.3.2 Intersection Sight Distance

Intersection sight distance must also be verified for any roundabout design to ensure that sufficient distance is available for drivers to perceive and react to the presence of conflicting vehicles, pedestrians, and bicyclists. At roundabouts, the only location requiring evaluation of intersection sight distance is at entry of the roundabout.

Intersection sight distance is achieved by establishing sight triangles where the triangle is bounded by a length of roadway defining a limit away from the intersection on each of the two approaches and by a line connecting those two limits. For roundabouts, these legs should be assumed to follow the curvature of the roadway, and thus distances should be measured not as straight lines but as distances along the vehicular path as shown in Figure 19-32.

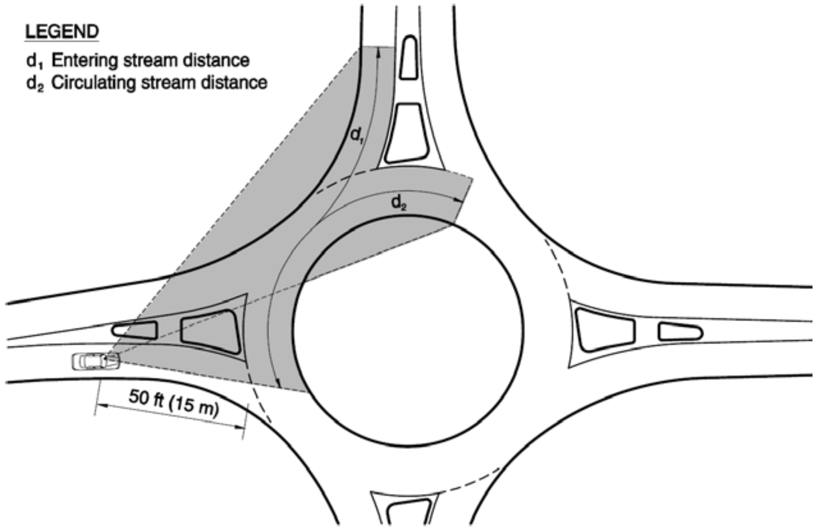


Figure 19-32 [NCHRP Report 672 Exhibit 6-58 (1)] Intersection Sight Distance

The approach leg of the sight triangle should be no more than 50 feet as shown in Figure 19-32. International research shows that excessive intersection sight distance can lead to higher vehicle speeds and a higher frequency of crashes. In most cases, it is best to provide no more than the minimum required intersection sight distance. Landscaping within the central island can be effective in restricting sight distance to the minimum requirements.

As shown in Figure 19-32, a vehicle approaching an entry to a roundabout faces two conflicting traffic streams; the entering stream of the immediate upstream entry (d_1) and the circulating stream (d_2). Vehicle speeds for the entering stream can be approximated by taking the average of the theoretical entering (R_1) speed and the circulating (R_2) speed. Vehicle speeds for the circulating stream can be approximated by taking the speed of the left-turning vehicles (R_4). The length of the conflicting leg is calculated using Equation 19-2 and Equation 19-3.

$$\boldsymbol{d_1 = (1.468)(V_{major,entering})(t_c)} \qquad [19\text{-}2]$$

$$\boldsymbol{d_2 = (1.468)(V_{major,circulating})(t_c)} \qquad [19\text{-}3]$$

where,

d_1 = length of entering leg of sight triangle, ft;

d_2 = length of circulating leg of sight triangle, ft;

V_{major} = design speed of conflicting movement, mph; and

t_c = critical headway for entering the major road, s, equal to 5.0 seconds

The critical headway for entering the major road is based on the amount of time required for a vehicle to safely enter the conflicting stream. The critical headway value of 5.0 seconds given in Equation 19-2 and Equation 19-3 is based upon the critical headway required for passenger cars. Table 19-4 shows computed length of the conflicting leg of an intersection sight triangle.

Conflicting Approach Speed (mph)	Computed Distance (ft)
10	73.4
15	110.1
20	146.8
25	183.5
30	220.2
Note: Computed distances are based on a critical headway of 5.0 s.	

Table 19-4 [NCHRP Report 672 Exhibit 6-59 (1)] Computed Length of Conflicting Leg of Intersection Sight Triangle

19.7.4 Angles of Visibility

The intersection angle between consecutive entries must not be overly acute in order to allow drivers to comfortably turn their heads to the left to view oncoming traffic from the immediate upstream entry. The intersection angle between consecutive entries, and indeed the angle of visibility to the left for all entries, should conform to the same design guidelines as for conventional intersections. Guidance for designing for older drivers and pedestrians recommends using 75° as a minimum intersection angle.

At roundabouts, the intersection angle may be measured as the angle between a vehicle's alignment at the entrance line and the sight line required according to intersection sight distance guidelines. Figure 19-33 illustrates an example where the angle of visibility is poor and the intersection needs to be improved. Figure 19-34 shows an example of a possible correction to improve the angle of visibility.

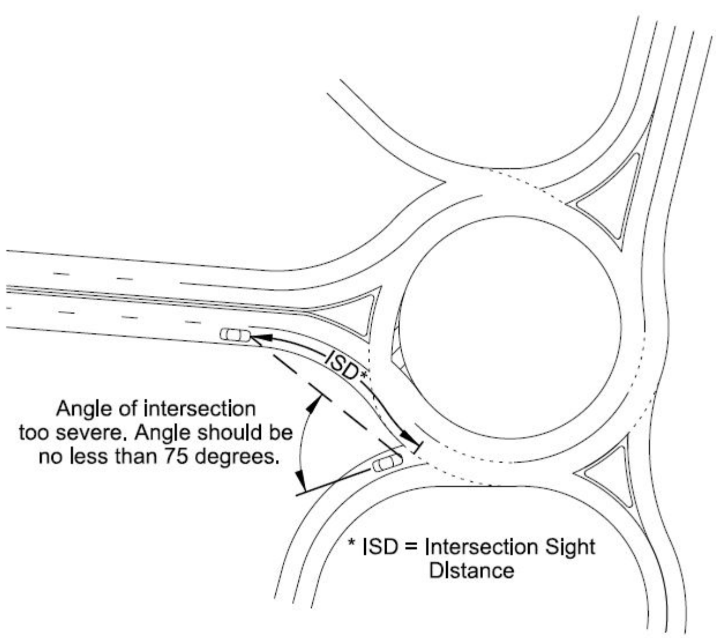


Figure 19-33 Roundabout Example with Poor Angle of Visibility

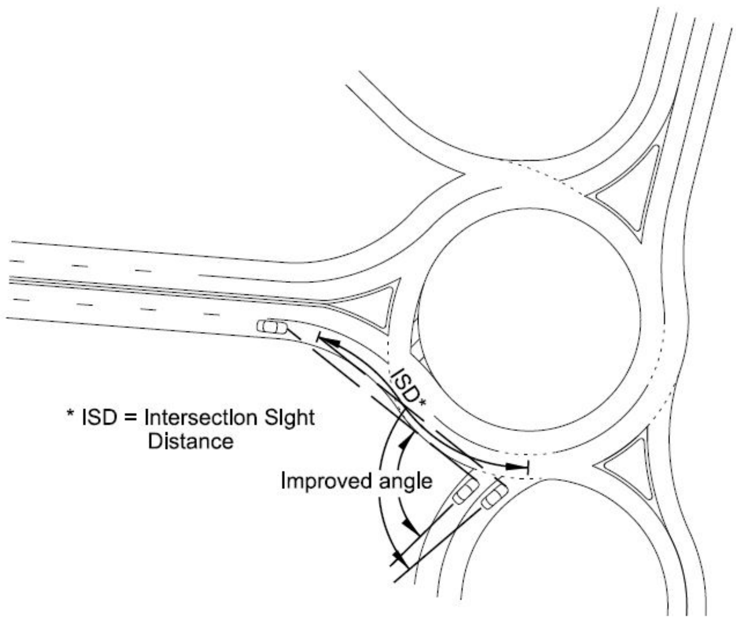


Figure 19-34 Roundabout Example with Improved Angle of Visibility

19.8 DESIGN DETAILS

The following are a general set of design detail guidelines to be considered at roundabouts. These are not to be interpreted as a standard or rule, or to be a complete set of design detail elements to considered. They are, however, a general set of best practices that the engineer should strive to achieve.

19.8.1 Sidewalk Considerations

Wherever possible, sidewalks at roundabouts should be set back from the edge of the circulatory roadway with a landscape strip, as shown in Figure 19-35. A recommended set back distance of 5 feet should be used (2-foot minimum), and it is best to plant low shrubs or grass in the area between the sidewalk and curb.

The recommended sidewalk width at roundabouts is 6 feet (5 feet minimum). In areas with heavy pedestrian volumes, sidewalks should be as wide as necessary to accommodate the anticipated pedestrian volume. At any roundabout where ramps provide sidewalk access to bicyclists, the sidewalk should be a minimum of 10 feet wide to accommodate shared use by pedestrians and bicyclists. Examples of sidewalk setback are shown in Figure 19-35 and Figure 19-36.

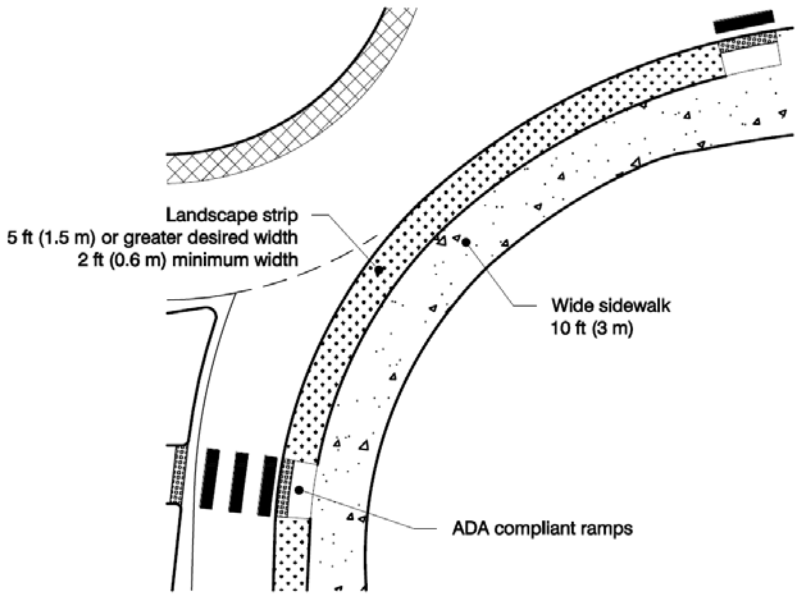


Figure 19-35 [NCHRP Report 672 Exhibit 6-63 (1)] Example Roundabout Sidewalk

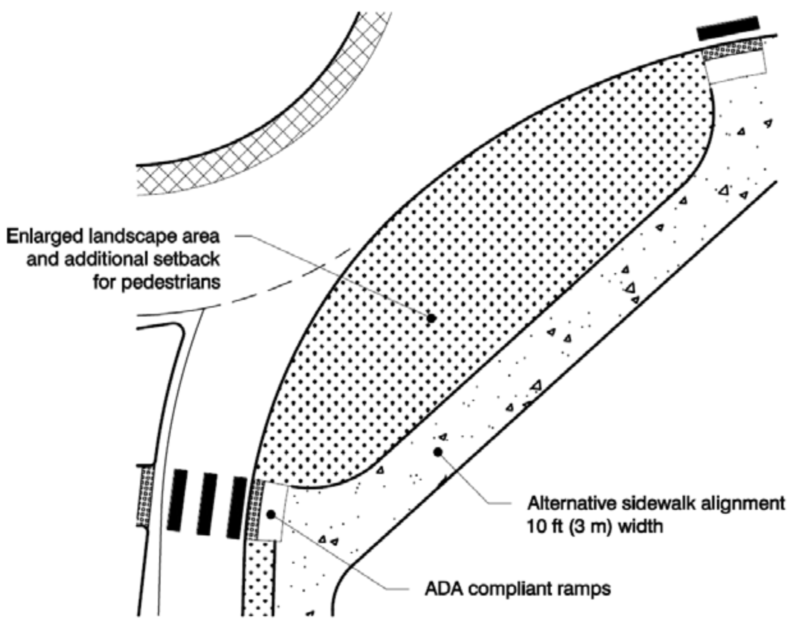


Figure 19-36 [NCHRP Report 672 Exhibit 6-64 (1)] Alternative Roundabout Sidewalk Treatment

19.8.2 Crosswalk Considerations

Pedestrian crosswalk placement at roundabouts requires consistency, based on a balance between pedestrian convenience, pedestrian safety, and roundabout operations. Pedestrian crosswalks should be designed as follows:

- The raised splitter island width should be a minimum of 6 feet at the crosswalk to adequately provide pedestrian refuge.
- A typical and minimum crosswalk setback of 20 feet is recommended (see Figure 19-7). This is the length of one vehicle without any additional distance to account for the gap between vehicles. At some roundabouts, it may be desirable to place the crosswalk two or three car lengths (45 feet to 70 feet) back from the edge of the circulatory roadway.
- The walkway through the splitter island should be cut-through instead of ramped. This is less cumbersome for wheelchair users and allows the cut-through walkway to be aligned with the crosswalks, providing guidance for all pedestrians, but particularly for those who are blind or who have low vision. The cut-through walkway should be approximately the same width as the crosswalk, ideally a minimum width of 10 feet.

Raised crosswalks (speed tables with pedestrian crossings on top) are another design treatment that can encourage slow vehicle speeds where pedestrians cross. Refer to Chapter 14 for additional information regarding pedestrian crossings at roundabouts.

19.8.3 Bicycle Design Considerations

When designing a roundabout, the engineer should provide bicyclists with similar options to negotiate roundabouts as they have at other intersections. Consider how they navigate either as motor vehicles or pedestrians depending on the size of the intersection, traffic volumes, their experience level, and other factors.

Bicyclists are often comfortable riding through single-lane roundabouts in low-volume environments in the travel lane with motor vehicles, as speeds are comparable and potential conflicts are low. At larger or busier roundabouts, cyclists may be more comfortable using ramps connecting to a sidewalk around the perimeter of the roundabout as a pedestrian. Roundabouts can be designed to simplify this choice for cyclists.

Where bicycle lanes or shoulders are used on approach roadways, they should be terminated at least 100 feet in advance of the circulatory roadway of the roundabout. Bicycle lanes should not be located within the circulatory roadway of roundabouts. Terminating the bike lane helps remind cyclists that they need to merge. At roundabout exits, an appropriate taper should begin after the crosswalk, with a dotted line for the bike lane through the taper. The solid bike lane line should resume as soon as the normal bicycle lane width is available.

Because some cyclists may not feel comfortable traversing some roundabouts in the same manner as other vehicles, bicycle ramps can be provided to allow access to the sidewalk or a shared use path at the roundabout. Bicycle ramps at roundabouts have the potential to be confused as pedestrian ramps, particularly for pedestrians who are blind or who have low vision. Therefore, bicycle ramps should only be used where the roundabout complexity or design speed may result in less comfort for some bicyclists. In general, bicycle ramps should not normally be used at urban, single-lane roundabouts, however, they may be appropriate if traffic speeds or other conditions (e.g., a right-turn bypass lane) make circulating like other vehicles more challenging for bicyclists.

Where bicycle ramps are provided at a roundabout, consideration should be given to providing a shared-use path or a widened sidewalk at the roundabout as discussed in 19.8.1. Bicycle ramps should be placed at the end of the full-width bicycle lane where the taper for the bicycle lane begins. Cyclists approaching the taper and bike ramp will thus be provided the choice of merging left into the travel lane or moving right onto the sidewalk. Bike ramps should not be placed directly in line with the bike lane or otherwise placed in a manner that appears to cyclists that the bike ramp and the sidewalk is the recommended path of travel through the roundabout.

Wherever possible, bicycle ramps should be placed entirely within the planting strip between the sidewalk and the roadway as shown in Figure 19-37. In these locations, the bicycle ramps should be placed at a 35° to 45° angle to the roadway and the sidewalk to enable cyclists to use the ramp even if pulling a trailer, but to discourage them from entering the sidewalk at high speed. The bike ramp can be fairly steep, with a slope potentially as high as 20%. If placed within the sidewalk area itself, the ramp slope must be built in a manner so that it is not a tripping hazard.

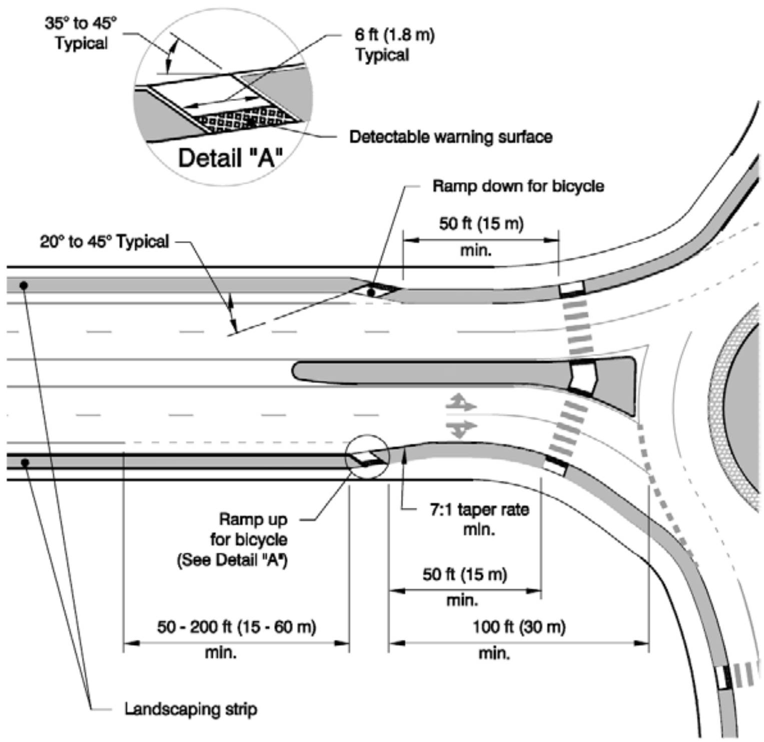


Figure 19-37 [NCHRP Report 672 Exhibit 6-67 (1)] Possible Treatments for Bicycles at Roundabouts

Since bike ramps can be confusing for pedestrians with vision impairments, detectable warnings should be included on the ramp. Where the ramp is placed in a planter strip, the detectable warning tile should be placed at the top of the ramp since the ramp itself is part of the vehicular area for which the detectable warning is used. If the ramp is in the sidewalk itself (as shown as one of the options in Figure 19-38), the detectable warning should be placed at the bottom of the ramp. Refer to Chapter 14 for additional information regarding bicycles at roundabouts.

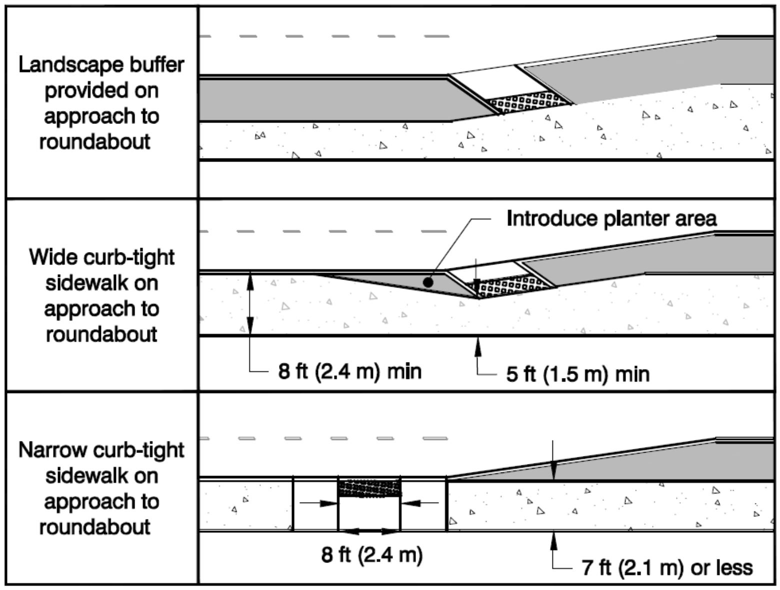


Figure 19-38 [NCHRP Report 672 Exhibit 6-68 (1)] Bicycle Ramp Design Options

19.8.4 Parking and Bus Stop Considerations

Parking in the circulatory roadway is not conducive to efficient and safe roundabout operations and should typically be prohibited. Parking on entries and exits should also be set back far enough so as not to hinder roundabout operations or to impair the visibility of pedestrians. AASHTO recommends that parking should end at least 20 feet from the crosswalk of an intersection. Curb extensions or bulb-outs are recommended to clearly mark the limit of permitted parking and reduce the width of the entries and exits.

For safety and operational reasons, bus stops should be located sufficiently far away from entries and exits and never in the circulatory roadway.

19.8.5 High-Speed Approach Considerations

An important feature affecting safety at rural intersections is the visibility of the intersection itself. Where possible, the geometric alignment of approach roadways should be constructed to maximize the visibility of the central island and the shape of the roundabout. Where adequate visibility cannot be provided solely through geometric alignment, additional treatments (signing, pavement markings, advanced warning beacons, etc.) should be considered.

On open rural highways, changes in the roadway’s cross section can be an effective means to help approaching drivers recognize the need to reduce their speed. Rural highways typically have no outside curbs with wide paved or gravel shoulders. Narrow shoulder widths and curbs on the outside edges of pavement, on the other hand, generally give drivers a sense they are entering a more controlled setting, causing them to naturally slow down. Thus, when installing a roundabout on an open rural highway, curbs should be provided at the roundabout and on the approaches, and consideration should be given to reducing shoulder widths.

Another effective cross-section treatment to reduce approach speeds is to use longer splitter islands on the approaches. Splitter islands should generally be extended upstream of the entrance line to the point at which entering drivers are expected to begin decelerating comfortably. A minimum length of 200 feet is recommended for high-speed approaches.

Another method to achieve speed reduction that reduces crashes at the roundabout while minimizing single-vehicle crashes is the use of successive curves (chicanes) on approaches, as shown in Figure 19-39. These approach curves should be successively smaller in order to minimize the reduction in speed between successive curves.

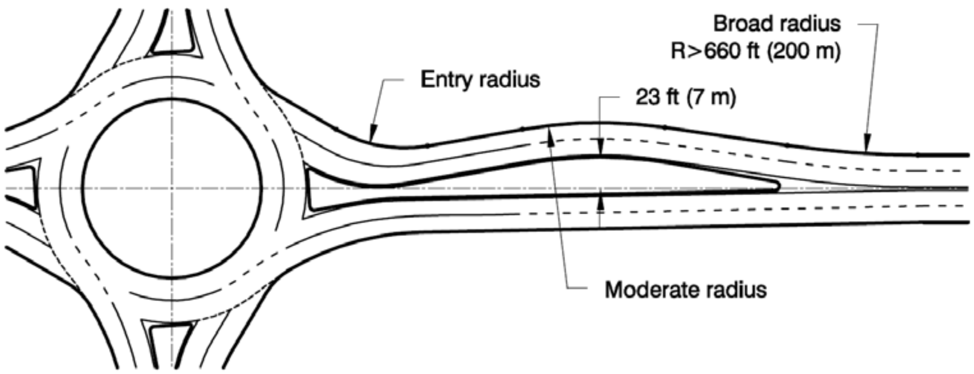


Figure 19-39 [NCHRP Report 672 Exhibit 6-40 (1)] Use of Successive Curves on High-Speed Approaches

19.8.6 Vertical Considerations

The vertical design of a roundabout begins with the development of the approach roadway and central island profiles. The development of each profile is an iterative process that involves tying the elevations of the approach roadway profiles into a smooth profile around the central island.

Each approach profile should be designed to the point where the approach baseline intersects with the central island as shown in Figure 19-40. A profile for the central island is then developed that passes through these four points (in the case of a four-legged roundabout). The approach roadway profiles are then readjusted as necessary to meet the central island profile.

It is generally not desirable to place roundabouts in locations where grades through the intersection are greater than 4%, although roundabouts have been installed on grades of 10% or more. At locations where a constant grade must be maintained through the intersection, the circulatory roadway may be constructed on a constant-slope plane. This means, for instance, that the cross slope may vary from +3% on the high side of the roundabout (sloped toward the central island) to −3% on the low side (sloped outward). On approach roadways with grades steeper than −4%, it is more difficult for entering drivers to slow or stop on the approach. At roundabouts on crest vertical curves with steep approaches, a driver's sight lines may be compromised, and the roundabout may violate driver expectancy.

Entry grade profiles (approximately two car lengths from the outer edge of the circulatory roadway) should not exceed 3%, with 2% being the desirable maximum. It is desirable to match the exit grades and the entry grades; however, the exit grade may be steeper but should not exceed 4%. Adjustments to the circulatory roadway cross slope may be required to meet these criteria but should be balanced with the effects on the circulatory roadway.

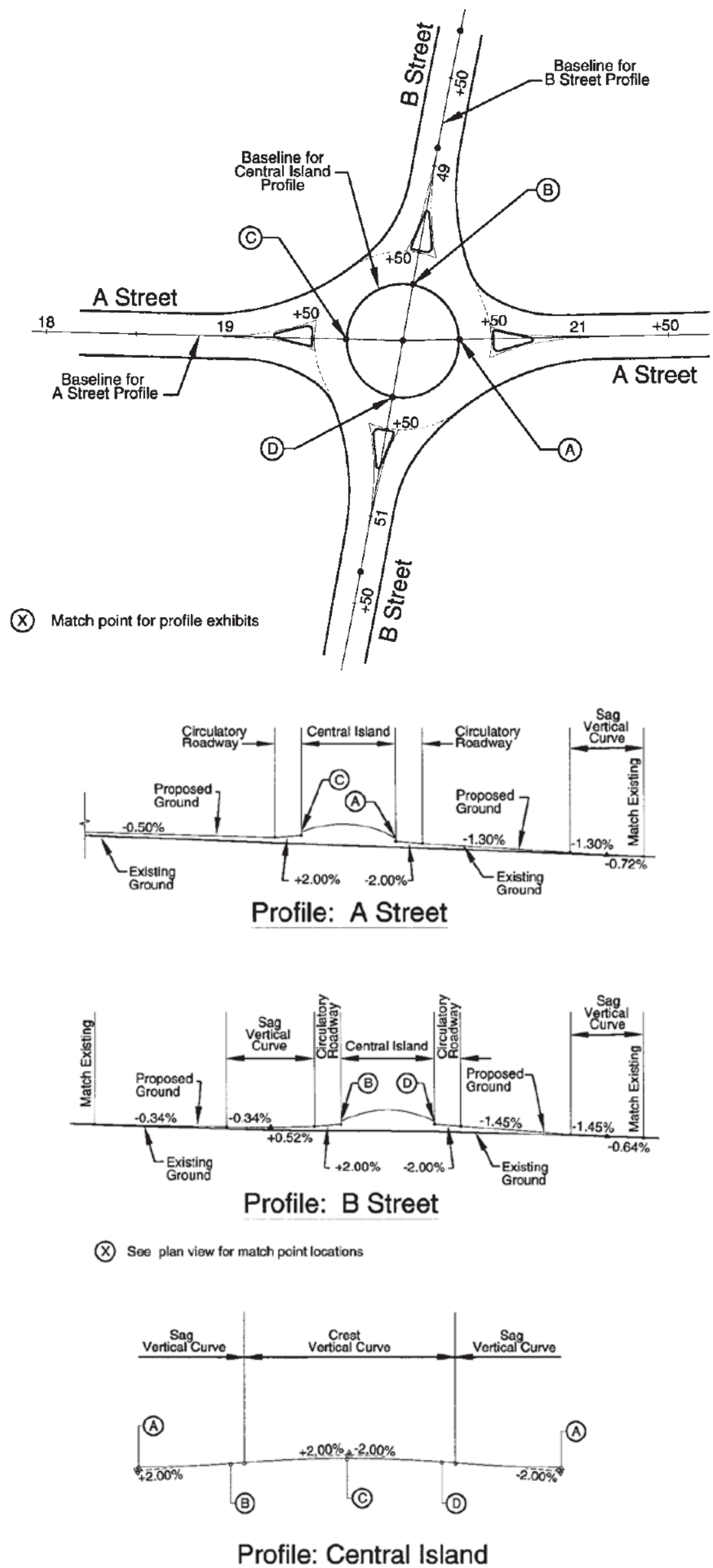


Figure 19-40 [NCHRP Report 672 Exhibit 6-75 (1)] Sample Central Island Profile

19.8.7 Cross Slope

As a general practice, a cross slope of 2% away from the central island should be used for the circulatory roadway on single-lane roundabouts. This is most practical in relatively flat terrain, however, roundabouts in hilly terrain may require the engineer to warp the profile to get the vertical design to work. It should be noted that excessive negative superelevation can result in an increase in single-vehicle crashes and loss-of-load incidents for trucks, particularly if speeds are high.

One method, primarily intended for consideration at multilane roundabouts, is to crown the circulatory roadway. The circulatory roadway is crowned with approximately two-thirds of the width sloping toward the central island and one-third sloping outward. This may alternatively be reversed so that half of the circulatory roadway slopes toward the central island. The maximum recommended cross slope is 2%. Asphalt paving surfaces are recommended under this type of application to produce a smoothed crown shape.

19.8.8 Truck Apron

Where truck aprons are used, the slope of the apron should generally be no more than 2%. Greater slopes may increase the likelihood of loss-of-load incidents. It is preferred to slope truck aprons away from the central island toward the outside of the roundabout, however, some locations have also implemented roundabouts with truck aprons sloped inward (toward the central island) to minimize water shedding across the roadway and to minimize load shifting in trucks.

The vertical design of the truck apron should be reviewed to confirm that there is sufficient clearance for low-boy type trailers, some of which may have only 6 to 8 inches between the roadway surface and bottom of the trailer. The vertical clearance can be reviewed by drawing a chord across the apron in the position where the trailer would sweep across. In some cases, warping of the profile along the circulatory roadway can create high spots that could cause trailers to drag or scrape along the truck apron. This should be checked during final design.

Between the truck apron and the circulatory roadway, a curb is required to accommodate a change in vertical elevation. As discussed in Section 19.4.4, the outer edge of the apron should be raised approximately 2 to 3 inches above the circulatory roadway surface. The apron should be constructed of a different material than the pavement to differentiate it from the circulatory roadway.

19.8.9 Drainage

With the circulatory roadway sloping away from the central island, inlets will generally be placed on the outer curb line of the roundabout. Inlets can usually be avoided on the central island for a roundabout designed on a constant grade through an intersection. As with any intersection, care should be taken to ensure that low points and inlets are placed upstream of crosswalks.

19.8.10 Concrete Jointing Patterns

If concrete pavement is used, joint patterns should be concentric and radial to the circulating roadway within the roundabout. Ideally the joints should not conflict with pavement markings within the roundabout, although concrete panel sizes may control this. On multilane roundabouts, circumferential joints within the circulating roadway should follow the lane edges to the extent practical.

19.8.11 Access Management

It is preferable to avoid locating driveways where they must take direct access to a roundabout. Nonetheless, site constraints sometimes make it necessary to consider providing direct access into a roundabout.

Public and private access points near a roundabout often have restricted operations due to the channelization of the roundabout. Driveways between the crosswalk and entrance line complicate the pedestrian ramp treatments and introduce conflicts in an area critical to operations of the roundabout. Driveways blocked by the splitter island will be restricted to right-in/right-out operation and are best avoided altogether unless the impact is expected to be minimal and/or no reasonable alternatives are available.

Queuing from nearby intersections (the roundabout or others nearby) should be checked to see if the operation of the access point will be affected.

19.8.12 Illumination

To improve the users' understanding of the roundabout's operations, the illumination should be designed to create a break in the linear path of the approaching roadway and emphasize the circular aspect of the roundabout

Illumination is recommended for all roundabouts, including those in rural environments. However, it can be costly to provide if there is no power supply in the vicinity of the intersection. Where lighting is not provided, the intersection should be well signed and marked (including the possible use of reflective pavement markers) so that it can be correctly perceived by day and night, recognizing that signing and markings alone cannot correct for the limited view of headlights when circulating.

In areas where only the roundabout is illuminated (no lighting is provided on the approach roadways), the scope of illumination needs to be carefully considered. Any raised channelization or curbing should be illuminated. A gradual illumination transition zone should be provided beyond the final trajectory changes at each exit. This helps drivers adapt their vision from the illuminated environment of the roundabout back into the dark environment of the existing roadway.

REFERENCES

1. NCHRP. *NCHRP 672, Roundabouts: An Informational Guide, 2nd ed. (NCHRP Guide 2)*, Transportation Research Board, National Academy Press, Washington D.C., 2010.

2. AASHTO. *A Policy on Geometric Design of Highways and Streets*, American Association of State Highway and Transportation Officials, Washington, D.C.: 2011.

CHAPTER 20
TRAFFIC AND SAFETY ENGINEERING

20.0 INTRODUCTION

The mission of CDOT's Traffic and Safety Engineering Branch is to reduce the frequency and severity of motor vehicle crashes which have a comprehensive and economic cost. This chapter addresses most, but not all, important design issues related to providing a safer transportation system.

20.1 ROADWAY SAFETY

Major factors affecting highway safety are the roadway and roadside features, driver ability and awareness, environmental factors, and vehicle characteristics. Highway safety is greatly influenced by variations among drivers (human factors). The drivers' knowledge and driving performance in a given environment or roadway condition are the primary determinants of safety. However, there are other factors such as highway design that also have a tangible impact on safety.

Design consistency in terms of geometry, cross section, and hazard shielding and mitigation should be maintained for entire corridors to minimize unexpected conditions. Advance warnings of changing conditions should be provided.

The designer should request a Transportation Systems Management & Operations (TSM&O) Evaluation from their Region Traffic Representative (RTR) for a safety analysis of their project which may lead to a more detailed safety assessment report. The report or analysis will make recommendations for safety improvements based on evaluation of the crash data within the project limits. The designer should document decisions to apply or not to apply any given safety feature in accordance with *CDOT Project Development Manual* **(1)**.

Primary references for safer designs are the AASHTO *Highway Safety Design and Operations Guide* **(2)**, and the *AASHTO Roadside Design Guide* **(3)**.

20.1.1 Bicycle and Pedestrian Safety

Bicyclists and pedestrians should be considered when scoping all projects. Bicycle and pedestrian traffic, shoulder width, and shoulder rumble strips should be addressed during the scoping stage of any project, including resurfacing projects.

See Chapter 14 of this Guide, "Bicycle and Pedestrian Facilities."

20.1.2 Railroad-Highway Grade Crossings

Railroad-highway grade crossings involve two distinct modes of transportation with different operating authorities and operating characteristics. Roadway and railway may intersect at-grade, or may be grade-separated by a structure that carries the roadway over or under the railroad. The majority of the nation's railroad-highway grade crossings remain at-grade. A railroad-highway grade crossing is typified by continuous vehicular traffic, interrupted periodically by a train's passage. The intermittent nature of train operations may dull a driver's awareness to a train's possible approach. Some drivers are tempted to disregard warnings and try to beat a train through the crossing. Except in unusual circumstances, trains have the right-of-way due to their huge mass, which often results in very long stopping distances. Safety at railroad-highway grade crossings is of utmost importance. The designer should include appropriate features to discourage risky driver behaviors, to provide sufficient advance notice of the grade crossing and of a train's approach or presence, and, as appropriate, to physically prohibit vehicles from entering the crossing.

Strategies for improving railroad-highway grade crossing safety include upgrading warning devices and improving the geometry, sight distance, and ride quality of the crossing. Active grade crossings contain train-activated devices that warn drivers of the approach or presence of a train. When new devices such as gate supports are installed, they may become roadside hazards and warrant shielding from errant vehicles.

Passive grade crossings lack such warning devices and rely on signs and pavement markings to identify the crossing location. Passive grade crossings have a higher risk for crashes because there is less direct control over driver actions. Where passive grade crossings remain in place, installation of enhanced sign systems may increase driver awareness and responsiveness.

Active railroad-highway grade crossings that are located adjacent to a signalized roadway intersection increase the complexity of signing and signals. Drivers may receive conflicting information from such closely spaced signals, or traffic stopped at the adjacent signalized intersection may queue back onto the grade crossing. Consideration should be given to interconnecting the traffic control signal with the active control system of the railroad crossing and providing a "pre-emption" sequence. With pre-emption, the approach of a train causes the traffic signals to enter a special mode to control traffic movements in coordination with the train's passage through the crossing. Traffic control signals near rail-highway grade crossings shall conform to Section 8C.09 of the *Manual on Uniform Traffic Control Devices for Streets and Highways (MUTCD)* **(4);** and pre-emption shall conform to Section 4D.27 of the *MUTCD* **(4).**

When a railroad-highway grade crossing is located within the limits of a rehabilitation project, the crossing, along with any existing devices, should be relocated or reconfigured as necessary to be compatible with changes to the highway. A safety assessment of the existing crossing should also be made and, to the extent feasible, the project should include any appropriate crossing safety improvements.

In 1991, the Federal Railroad Administration established a goal of closing 25 percent of all railroad-highway grade crossings in the country. Closing unnecessary grade crossings improves safety by eliminating the potential for vehicle-train crashes and by concentrating limited safety funds on the remaining crossings. Guidance for eliminating and consolidating railroad-highway grade crossings is provided in AASHTO *Highway-Rail Crossing Elimination and Consolidation* **(5).**

All projects involving work on railroad property or adjustments to railroad facilities require a written contract among CDOT, the railroad, and any involved local agencies. Any changes to a grade crossing's operating characteristics should also be coordinated with the Colorado Public Utilities Commission (PUC). The CDOT Railroad and Utilities Program within the Project Development Branch administers the highway-rail grade crossing program and is the Department's point of contact with the railroad, the PUC, and local agencies on all CDOT railroad contracts.

For additional guidance on railroad-highway grade crossing components, safety assessment, safety measures, project development, and traffic control, see:

- FHWA *Railroad-Highway Grade Crossing Handbook* **(6)**.
- FHWA *Guidance on Traffic Control Devices at Highway-Rail Grade Crossings* **(7)**.
- *Manual on Uniform Traffic Control Devices (MUTCD)* Part 8 **(4)**.
- CDOT *Railroad Manual* **(8).**

20.1.3 Roadway Geometry Considerations

Horizontal curves typically have crash rates from 1.5 to 4 times higher than tangent sections. Crash rates tend to increase with the reduced sight distance associated with either a reduced curve radius or an increased deflection angle. Therefore, it is usually beneficial to maximize curve radii and minimize deflection angles when designing alignments. The use of spiral curves can help to mitigate some of the safety problems associated with horizontal curves by:

- Providing a safer path for the driver from a tangent position to a curve position
- Providing a location for the required length of transition from normal crown to full superelevation.

The attributes of spiral curves are explained in section 3.2.2.2 of this Guide and Chapter 3 of *PGDHS* **(9)**.

Vertical curves can also lead to higher crash rates due to the reduced sight distance imposed by the crest of a vertical curve. Accordingly, the designer should minimize the severity of vertical curvatures in the alignment design. Intersections located on or near vertical curves should be investigated thoroughly and avoided when practical alternatives can be found.

20.1.4 Intersections

Intersections are the major points of conflict in roadways. Safety measures that can reduce conflict, particularly from left turns, include medians, protected left-turn phasing of signals, auxiliary lanes, among other measures. Left-turn lanes should be designed with an offset to provide for proper sight distance to oncoming traffic when a vehicle is in the opposing left-turn lane. Safer methods for accommodating pedestrian and bicycle traffic movements through the intersection should also be considered (see Chapter 14). All-way stops or roundabouts may sometimes be desirable alternatives to traffic signals. Criteria for all-way stops are found in Section 2B of the *(MUTCD)* **(4)**. Further information on roundabout design is found in Chapter 19 of this Guide and FHWA's *Roundabouts: An Informational Guide* **(10)**.

"Stop Ahead" warning signs should be placed ahead of intersections where the driver may not anticipate the required stop or where sight distance is obstructed. For additional emphasis, a yellow beacon above the "Stop Ahead" sign, a red beacon above the "Stop" sign, or both can be considered.

The designer must consider the corner radii and sight distances at intersections. A large corner radius can increase the roadway crossing width, making it difficult for pedestrian crossing, and will allow for increased vehicle speed around intersection corners. Conversely, too small a radius can cause maneuverability problems for large vehicles, potentially adding to pedestrian conflicts.

As part of a project, traffic signals should be rebuilt or modified to comply with current standards and to meet design traffic demands.

It is essential to have proper application of traffic control devices to designate right of way and safe movement of all traffic, including pedestrians and bicyclists. Follow the standards published in the latest edition of the *MUTCD* **(4)**.

20.1.5 Interchanges

Interchanges do not always have the same direct conflict potential as intersections, but the vehicle merge areas often show a higher frequency of crashes.

A large portion of truck crashes occurs at interchanges. The potential for overturning of high profile vehicles increases on circular ramps. Adequate signing, careful attention to merging patterns, and ramp geometrics can mitigate these problems. These concepts are detailed in the *Highway Capacity Manual* **(11)**. See chapter 10 for interchange design.

20.1.6 Context Sensitive Solutions

Safety is a challenge to be addressed on every project. It may not be the primary driver for a given project, but it should be a consideration in the development and evaluation. Context Sensitive Solutions (CSS) equally address safety, mobility, and the preservation of scenic, aesthetic, historic, environmental, and other community values. To balance these values the design process should be flexible in adherence to standards and criteria. A successful context- sensitive solution produces transportation designs that address both safety and feasibility. CSS maintains safety and mobility as priorities, yet recognizes that these are achieved in varying degrees with alternative solutions. Utilizing the CSS philosophy, CDOT design professionals determine which solution best fits, given the site's conditions and context. CSS is about making good engineering decisions.

NCHRP Report 480, *A Guide to Best Practices for Achieving Context Sensitive Solutions (CSS)* **(12)** discusses two types of safety: nominal safety and substantive safety. “Nominal safety” equates adherence to standards or design policy with achieving safety, and considers substandard designs to be unsafe. Under the nominal safety concept, a roadway designed to current or modern criteria would be characterized as ‘safe’. However, engineers should also consider substantive safety in the design process. Substantive safety refers to the actual (or expected) crash frequency and severity for a highway or roadway.

These two types of safety should be considered when addressing a safety problem. The solution should balance cost, environment, and other stakeholder values. High-crash locations with substandard design features should be prioritized for improvement. Locations that are nominally safe, but substantively less safe also should be considered.

For every project, the setting and character of the location, the values of the community, and the needs of highway users should be balanced.

Consider the following:

- Flexibility provided within the standards.
- Design exceptions where there are environmental concerns.

- Opportunities to re-evaluate decisions made in the planning phase.
- Design speed.
- Preservation of existing horizontal and vertical geometry, cross section, and design for resurfacing, restoration, and rehabilitation (3R) improvements where it is known that safety problems do not exist.
- Alternative standards for a corridor or scenic route.
- Safety and operational impact of various design features and modifications.

These concepts are discussed in detail in NCHRP Report 480, A Guide to Best Practices for Achieving Context Sensitive Solutions (CSS) **(12)** and Flexibility in Highway Design **(13)**. Additional information on context-sensitive solutions can be found in the following references:

- NCHRP Report 480, *A Guide to Best Practices for Achieving Context Sensitive Solutions* **(12)**
- NCHRP Report 374, *Effect of Highway Standards on Highway Safety* **(14)**
- NCHRP Project 430, *Improved Safety Information to Support Highway Design* **(15)**
- NCHRP Report 362, *Roadway Widths for Low-Traffic-Volume Roads* **(16)**
- IHSDM: *Interactive Highway Safety Design Model* **(17)**
- FHWA, *Prediction of the Expected Safety Performance of Rural Two-Lane Highways* **(18)**
- CDOT Chief Engineer's Policy Memo 26, *Context Sensitive Solutions (CSS) Vision for CDOT* **(19)**

20.1.7 Work Zone Safety

Proper traffic control, delineation, and channelization are critical to achieving safety in work zones. A work zone can pose additional hazards to the motorist and cause risk to workers. All traffic control devices must meet the guidelines in the *AASHTO Manual for Assessing Safety Hardware (MASH)* **(40),** *NCHRP Report 350, Recommended Procedures for the Safety Performance Evaluation of Highway Features* **(20)** and the *MUTCD* **(4)**. See also section 20.5.1.

20.1.8 Roadway Shoulders

Refer to CDOT *Policy Directive 902.0*, Shoulder Policy **(21)**.

20.2 REDUCING RUN-OFF-THE-ROAD CRASHES

Apply the following improvements where appropriate to reduce the frequency and severity of run-off-the-road crashes:

- Removing obstacles from the roadside
- Redesigning the obstacle
- Relocating obstacles from the clear zone
- Installing breakaway devices that reduce impact severity [AASHTO *MASH* **(40)** and *NCHRP Report 350* **(20)**]
- Shielding obstacles with guardrail
- Improving delineation
- Cable rail installation
- Upgrading guardrail
- Using rumble strips

- Applying Textured shoulder treatment
- Eliminating shoulder drop-offs, using safety edge technology
- Correcting superelevation
- Improving the pavement condition
- Improving the roadway geometry
- Flattening slopes
- Maintaining the clear zone

20.2.1 Rumble Strips

Studies have shown that rumble strips can reduce the frequency of run-off-the-road crashes. Rumble strips alert drivers when their vehicles stray onto the shoulder or over the centerline of the roadway. Rumble strips can also provide protection for pedestrians and bicyclists on the shoulder by discouraging motorists from straying onto the shoulder and provide an audible notice to pedestrians and bicyclists. Improperly installed rumble strips can force the bicyclist into the travel lane causing conflict with the motorists.

20.2.1.1 General Criteria

See Standard Plan M-614-1 of the *CDOT Standard Plans – M & S Standards* **(22)** for rumble strip details.

To maximize a smooth shoulder surface suitable for bicycle use, rumble strips should be installed adjacent to the edge of the travel lane per *CDOT Standard Plan M-614-1* **(22)**. AASHTO considers a 4-foot width on the shoulder beyond the rumble strip to be the minimum for safe bicycling. See AASHTO's *Guide for the Development of Bicycle Facilities* **(23)**.

Rumble strips should be used on rural highways at locations where run-off-the-road type crashes are most likely to occur. These locations should include:

- On long tangents.
- At approach ends of isolated horizontal curves.
- Along steep fill slopes.
- At approaches to narrow bridges.
- At documented high-crash locations.

Rumble strips should not be used where guardrail is installed on shoulders that are less than 6 feet wide. When rumble strips are discontinued for guardrail or narrow shoulders, the rumble strip should end at least 250 feet prior to the end section of the guardrail or the narrowing of the shoulder. This will allow bicyclists room to reposition their bikes on the shoulder.

Rumble strips are not normally used in urban areas because of the noise they cause and the frequent use of the roadway shoulder for turning or parking.

When warranted by crash history, centerline rumble strips may be used to mitigate head-on, sideswipe opposite, and opposite side run-off-the-road crashes in areas with a history of these types of crashes, mountainous areas, or areas where sight distance is constrained. When used, centerline rumble strips should be installed in "no passing" zones, but may continue into "passing" zones.

Shoulder rumble strips are somewhat less effective in mountainous areas or on roadways with a high-frequency of horizontal curves where drivers are generally more attentive. Rumble strips may be omitted on steep, downhill sections to provide bicyclists with more maneuvering room, particularly where run-off-the-road crash history is low.

20.2.1.2 Installation on Interstate Highways

Rumble strips should be installed on the inside (left) shoulders of all rural Interstate highways as shown on Standard Plan M-614-1 of the *CDOT Standard Plans – M & S Standards* **(22)** and may be continuous as determined by the designer.

They should be installed on the outside (right) shoulders providing the shoulder width is 6 feet or greater.

20.2.1.3 Installation on Narrow Shoulders

Where the system-wide evaluation indicates a significant history of run-off-the-road crashes, rumble strips may be considered if bicycle traffic can still be accommodated. Consider applying rumble strips only in high-crash locations rather than over the entire length of the roadway.

Before installing rumble strips on narrow shoulders, the designer should weigh the benefits to motorists, versus the reduction in usable bicycle riding width. Installation of rumble strips on shoulders which are 4 feet wide or narrower will provide bicycles with less than the AASHTO recommended 4-foot clear bike path and will have a negative impact on bicycle travel.

For further information on rumble strips, refer to the FHWA Rumble Strip Web Page **(24)** and to NCHRP *Synthesis 191 Use of Rumble Strips to Enhance Safety* **(25)**.

20.3 ROADSIDE SAFETY

Roadside safety is improved by reducing the likelihood of a vehicle leaving the roadway and by reducing the hazards faced by an errant vehicle that leaves the roadway. This section discusses the methods and tools used to improve roadside safety. Additional strategies can be found at the joint AASHTO-NCHRP web site for implementing the *NCHRP Project 17 – 18,* Strategic Highway Safety Plan **(26)**.

CDOT has adopted the *AASHTO Roadside Design Guide* **(3)** for use in determining barrier warrants, length of needed barrier, and overall roadside design considerations. Some of the items that are covered are:

- Barrier types and characteristics.
- Methods for mitigation of obstacles.
- Clear zone concept.
- Embankments and cut slopes.
- Fixed objects.
- Shoulder drop-offs.

CDOT Standard Plans - M & S Standards **(22)** contain design and typical installation details for guardrail, end treatments and transitions. The guardrail details on the *Standard Plans* do not fit all situations. A new item or design adaptation not covered by CDOT *Standard Plans - M & S Standards* **(22)** is not necessarily precluded from use. Consult the *Roadside Design Guide* **(3)** or contact the Standards and Specifications Unit in Project Development for additional information.

See the *CDOT Crash Cushion and End Treatment Selection Guide* **(27)** which is on the CDOT web site and contact the Standards Engineer to determine the acceptability of any alternative design.

20.3.1 Unique Hazards

Special situations may occur where protection is desirable even though not required; for example, where there is a potential obstacle that is not within the clear zone, or where there are objects with historic, environmental or economic significance.

20.3.2 Guardrail

Guardrail should be installed only at specific locations where roadside hazards warrant, and after all other possible mitigation measures have been considered. CDOT uses two primary types of guardrail: strong-post W-Beam (Type 3) and F-shaped concrete barrier (Type 7). See M&S Standards. Modified Thrie-Beam (Type 6), 3- and 4-strand cable guardrail, and other types are also used in special situations. A fully functional guardrail installation will consist of a transition (if changing rail rigidities), a run of computed length of need, and end treatments.

Refer to section 4.9 of this Design Manual for further information on guardrail.

20.3.2.1 Review of Crash History

For 3R projects, guardrail may be warranted in locations where there is a history of frequent run-off-the-road crashes. At least three years (but preferably five years) of the most recent crash data should be analyzed to determine if there is a need for guardrail.

20.3.2.2 Maintaining Continuity

Driver expectation is often a key component in determining guardrail placement. Consider how the placement of guardrail will affect the driver's perception of both the area where the guardrail is placed and the surrounding areas. Maintaining continuity of roadside characteristics is important and can affect the designer's guardrail decisions in many ways. Guardrail choices made for the first section of a corridor will affect the options available for guardrail in the subsequent sections. A decision should be made early in the scoping process on how the corridor will be designed to create a consistent type of roadway.

If a proposed guardrail installation is only marginally warranted, but the rest of the section has guardrail, then installing the guardrail may be appropriate. Placing guardrail, widening shoulders, or straightening horizontal curves may not be advisable for short sections of roadway when it will likely cause a motorist to exceed the safe operating conditions of adjacent segments yet to be improved. Improving safety in a corridor may sometimes be done in short sections, but the overall corridor safety should be maintained during the process. If isolated segments of a corridor are upgraded, a letter outlining the decision should be included in the project file.

20.3.2.3 Determination of Length

The procedure for determining the length of need for guardrail is contained in the *AASHTO Roadside Design Guide* **(3)**.

20.3.2.4 Offset

Standard Plans M-606-1 of the *CDOT Standard Plans – M & S Standards* **(22)** lists recommended offsets. As a general rule, if the shoulder width is 6 feet or less, the guardrail should be offset an additional 2 feet from the edge of shoulder. If the shoulder width is 8 feet or greater, no additional offset is required. The 2-foot offset is intended to provide additional width for opening the door of a parked or stranded vehicle.

In most cases, new guardrail should not be installed on the z-slope or side slope unless the slope is 10:1 or flatter. Where necessary, installations may be made on slopes as steep as 6:1, but only if they are located so that the errant vehicle is in its normal attitude at the moment of impact.

In general, the placement recommendations shown in the *AASHTO Roadside Design Guide* **(3)**, Table 5.5, should be followed.

20.3.2.5 Access Treatments

Short gaps between guardrail sections should always be avoided. Such gaps may allow vehicles to pass behind the rail or strike end treatments, which will cause greater damage than impacting the rail.

Short gaps should be addressed when designing access treatments. Some rules to follow include:

- Move accesses, if possible, to avoid gaps in guardrail.
- Remove obstacles around accesses (flatten slopes, relocate mailboxes, etc.).
- Install Type 3J End Anchorage and 3K Terminal, provided obstacles are cleared behind the rail (see details in the *CDOT Standard Plans - M & S Standards)* **(22)**.
- Install standard Type 3 guardrail with appropriate end treatments.
- Install Type 3 guardrail with reduced post spacing (see detail in *CDOT Standard Plans - M & S Standards*) **(22)**.

20.4 TRAFFIC ENGINEERING PLANS

Traffic control plans should include a “Schedule of Construction Traffic Control Devices,” construction traffic control plans, detour routes, temporary as well as permanent signing, striping, pavement markings, and signal plans.

20.4.1 Source Documents

Many documents and manuals govern the manner in which a set of traffic plans is prepared.

While the list below includes the main sources of information for the traffic engineer, it is not exhaustive. Traffic control and operations is an ever-changing field of engineering and the use of the latest state-of-the-art techniques is encouraged. See also the references at the end of this chapter.

- AASHTO *Highway Safety Design and Operations Guide* **(2)**.
- AASHTO *Roadside Design Guide* **(3)**.
- CDOT *Standard Plans* (“S” Standards, which are a part of the M & S Standards) **(22)**.
- *Colorado Supplement to the Standard Highways Signs* **(28)**.
- *Manual on Uniform Traffic Control Devices for Streets and Highways (MUTCD)* **(4)**.
- CDOT *Recommended Pavement Marking Practices* **(29).** Copies of this guideline are available from the Safety and Traffic Engineering Section.
- FHWA *2012 Supplement to Standard Highway Signs* **(30)**.
- FHWA *Standard Highway Signs* **(31)**.
- CDOT *Standard Specifications for Road and Bridge Construction* **(32)**.
- AASHTO *Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals* **(33)**.
- *Colorado Supplement to the MUTCD* **(34)**. Sets forth additions, deletions or changes to the *MUTCD* required by the peculiarities of Colorado State Law.
- ITE *Traffic Control Devices Handbook* **(35)**.
- ITE *Traffic Engineering Handbook* **(36)**.
- ITE *Transportation Planning Handbook* **(37)**

20.5 CONSTRUCTION TRAFFIC CONTROL

20.5.1 Construction Traffic Control Plan

The Construction Traffic Control Plan (TCP) is a strategy for safely moving traffic through a work zone. The Safety and Traffic Engineering Branch provides standards to be used for developing the TCP.

The components of a typical TCP are:

- Schedule of Construction Traffic Control Devices/Tabulation of Traffic Engineering items.
- Construction Signing Plan.
- Detour Routes.
- Tabulation of Signs.
- Permanent/Existing Signing Plan.
- Cross sections at Class III and overhead sign locations.
- Standard Overhead Sign Bridges/Standard Overhead Sign Cantilever/Standard Overhead Sign Butterfly.
- Tabulation of Pavement Markings.
- Pavement Marking Plan
- Signal Plan.
- List of Standard Special Provisions.
- List of Project Special Provisions.
- Detailed Sign Layouts.

Information contained in a TCP typically includes:

- Placement and maintenance of traffic control devices.
- Methods and devices for delineation and channelization.
- Construction scheduling.
- Application and removal of pavement markings.
- Roadway construction lighting requirements.
- Traffic regulations.
- Uniformed traffic control (surveillance).
- Inspection activities.

The TCP should be developed during the initial planning stages of any scheduled activity and should be considered in all decisions related to the activity. The Region Traffic Engineering Section will work closely with the Project Manager to develop a sound TCP for all construction activities. The TCP is included in the Contract Plan Package along with the specifications for the project.

The *MUTCD* **(4)** and *CDOT Standard Plans – M & S Standards* **(22)** provide a framework to develop a sound and effective TCP for all construction projects. See section 20.3 of this guide and section 3.10 “Noise Analysis” of the *CDOT Project Development Manual* **(1)**.

20.5.2 Construction Signing and Striping

Construction signing is an essential and integral part of any highway construction project. Part 6 of the *MUTCD* **(4)** and the “S” Standards of the *CDOT Standard Plans – M & S Standards* **(22)** provide examples of typical construction signing, methods of erection and signing placement to address a variety of typical construction activities. Construction signs are typically placed on the roadway for a short period of time, therefore avoiding the need for standard durable panel material. Section 630 of the *CDOT Standard Specifications for Road and Bridge Construction* **(32)** governs the choice of construction sign panel material.

The typical construction signing placement presented in the *MUTCD* **(4)** and Standard Plan S-630-1 of the *CDOT Standard Plans – M & S Standards* **(22)** and typical striping layout presented in Standard Plan S-627-1 of the *CDOT Standard Plans – M & S Standards* **(22),** are designed to assist those involved with construction traffic control, but are not intended to replace sound engineering judgment or the experience of a qualified traffic engineer.

20.5.3 Temporary Pavement Markings

Proper temporary striping is a key component of highway projects, particularly for delineation of passing and no-passing zones. Temporary pavement markings are used to supplement drums or traffic cones in a construction work zone or as provisional markings on a roadway. Temporary markings may be categorized as “Full-Compliance,” “Interim” or “Control Points.” Full Compliance markings are those meeting all the requirements of Part 3 of the *MUTCD* **(5)**.

When appropriate, interim markings, such as paint or removable pressure sensitive tape, are used until full-compliance markings are installed. Control points are placed for the purpose of guiding the installation of interim or full–compliance pavement markings.

In work zones where traffic is redirected for more than one-day, temporary pavement markings are typically placed along tapers and tangents, but may be placed elsewhere in the project if the need arises. Temporary pavement markings may be white or yellow depending on the type of marking (i.e., edge line, lane line or channelizing line) they replace. When construction is completed, temporary pavement markings should be easy to remove without damaging or scarring the roadway surface. In most cases, temporary pavement markings shall be removed and full-compliance markings installed within 14 days of completion of the project.

Estimates for temporary pavement marking quantities, whether they are paint or removable tape, are itemized on the Tabulation of Traffic Engineering Items plan sheet.

20.5.4 Channelizing Devices

Channelizing devices are designed to warn drivers of potential obstacles created by construction or maintenance operations on or near the traveled way, to protect workers in the work zone, and to guide and direct drivers and pedestrians safely past potential obstacles. These devices may be used to provide a smooth and gradual transition in moving traffic from one lane to another, onto a bypass or detour or in reducing the width of a lane. Channelizing devices should always be preceded by a system of warning devices adequate in size, number, and placement for the roadway. Channelizing devices should be designed in a way that minimizes damage to vehicles that inadvertently strike them.

Taper design is one of the most important elements within the system of construction traffic control devices. A poorly designed taper will almost always produce undesirable traffic operations, congestion, or possibly crashes. Tapers may be necessary in both the upstream and downstream directions of traffic depending on the construction activity. Tapers are classified as merging tapers, shifting tapers, shoulder tapers and two-way traffic tapers. Examples of tapers and formulas for calculating their minimum desirable lengths are found in the Standard Plan S- 630-1 of the *CDOT Standard Plans – M & S Standards* **(22)**.

A variety of channelizing devices have been approved by CDOT for use in construction projects. These channelizing devices include:

- Traffic cones
- Tubular markers
- Vertical panels
- Drums
- Barricades
- Concrete barriers
- Water-filled barriers

Traffic cones are typically reserved for lane closures and other construction activities during daylight hours. Traffic cones with retroreflective bands are also allowed for nighttime use, but only during working hours. The remaining channelizing devices listed above have been approved for both day and nighttime construction activities. Details regarding the placement of channelizing devices can be found in the *MUTCD* **(4)** and in the *CDOT Standard Plans – M & S Standards* **(22)**.

Quantities for all channelizing devices required on a construction project are tabulated in the Schedule of Construction Traffic Control Devices.

20.5.5 Special Devices

Other special traffic control devices may include variable message signs (VMS) and arrow panels. Requirements for the use of these devices are addressed in Part 6 of the *MUTCD* **(4)**.

20.5.6 Construction Staging/Phasing

Most highway construction projects require the maintenance of traffic throughout the work zone. The Region Traffic Engineering Section will work closely with the design and construction engineers to develop a construction staging concept that can expeditiously complete the project while safely and efficiently conveying traffic through the work zone. Construction signing plans should detail the construction signing schemes for all the planned phases of the project.

When appropriate, consider full road closures for construction projects to expedite construction and eliminate construction and traffic conflicts.

20.5.7 Construction at or Near Railroad-Highway Grade Crossings

Highway construction at or near railroad-highway grade crossings may require special traffic control measures to preserve highway and traffic safety, protect workers, and provide for the safe passage of trains through the project work zone. Construction traffic control activities involving railroads may occur on:

- Railroad-highway grade crossing safety projects.
- Other projects requiring work on or near railroad tracks or property.
- Railroad-highway grade separation structure projects.

Refer to the *MUTCD* Section 6G.18 **(4)** for standard guidance for work in the vicinity of grade crossings; and *MUTCD* Figure 6H-46 **(4)** for typical application of construction traffic control devices at grade crossings. It is necessary to prevent vehicles from stopping on tracks, and to prevent the queuing of stopped vehicles across the tracks.

Highway projects involving work on or near railroad tracks or crossings may, in addition to necessary traffic control measures at grade crossings, also require the use of railroad flaggers. Railroad flaggers are railroad employees who are authorized to stop or direct train traffic on the affected tracks. Whenever the highway work may pose a danger to trains or interfere with normal train movements (construction equipment near tracks, bridge demolition work, etc.), the railroad company will require a railroad flagger to be stationed at the project site. The flagger will monitor site conditions and exert positive control over trains passing through the project.

Railroad flagging requirements, if any, will be set forth in the project special provisions, and flaggers will be paid out of project funds in accordance with the special provisions. Railroad flagging rates (daily or hourly) will be specified by the railroad company.

Highway construction on railroad overpass structures may also require the use of railroad flaggers to guard against hazards to trains such as falling debris, bridge falsework, or construction equipment.

The required contract (see section 20.1.2) among CDOT, the railroad, and involved local agencies will set forth traffic control responsibilities, coordination requirements, and railroad flagging requirements. The designer should request a contract from the Project Development Branch well in advance of planned construction to allow sufficient time for contract development and execution. CDOT field construction personnel should closely coordinate traffic control with railroad and local agency representatives.

20.6 PERMANENT SIGNING

20.6.1 Uniform Standard Regulatory and Warning Signs

CDOT has adopted the *MUTCD* **(4)** guidelines for the placement of permanent regulatory and warning signs on the State highways. Signing shall be in conformance with the *MUTCD* Parts 2 and 3 **(4)**. Proper installation and consistency of signs provide guidance and information to safely travel a section of roadway. Signs should be clear and positioned for adequate response time, particularly on high-speed roadways. Detailed layouts and standard sizes for these signs can be found in the FHWA *Standard Highway Signs* **(31).** For further details including ground sign placement, consult the *MUTCD* **(4)** and Standard Plan S-614-1 of the *CDOT Standard Plan – M & S Standards* **(22)**. All signs must meet the *AASHTO Manual for Assessing Safety Hardware (MASH)* **(40)** and *NCHRP Report 350* **(20)** requirements for crashworthiness. See applicable "S" Standards in the *CDOT Standard Plan – M & S Standards* **(22)** and CDOT Specifications for currently accepted sign designs.

The Tabulation of Signs sheet provided for permanent signing on the project lists the panel sizes, post lengths, sign locations and color, *MUTCD* code **(4)**, foundation requirements, and quantities required on a construction project.

Signs should be replaced on a project when damaged, faded or no longer meet retroreflective requirements. For most new construction or reconstruction projects, signs should be updated or replaced. The designer should check with the Region Maintenance or Traffic Engineering Section for the replacement schedule. For overlay projects, the designer should examine the condition of existing signs to determine if replacement is needed. Signs that are more than ten years old will usually require replacement.

Signing is used for a wide range of purposes. The designer will follow the "S" Standards in the *CDOT Standard Plan – M & S Standards* **(22)** and the *MUTCD* **(4)** when determining the signing requirements for a project.

20.6.2 Special Signs

Special signs are those not designated with a sign code in the *MUTCD* **(4)**. These signs may include construction signs indicating detours or hours of operations or permanent signs such as guide signs, specific information signs, or other special interest signs. The Region Traffic Section will provide detailed sign layouts for all special signs. Legends shall consist of either upper or lowercase characters provided in the *2012 Supplement to Standard Highway Signs* **(30)**, with letter sizes following the guidelines in Part 2 of the *MUTCD* **(4)**.

Special signs are tabulated on the Schedule of Construction Traffic Control Devices or the Tabulation of Signs provided in each contract plan package.

Only symbols that have been approved by FHWA may be used on special signs.

20.6.3 Sign Classifications

Permanent sign panels placed on the State highway system are classified as Class I, II or III.

Class I sign panels are single-sheet aluminum with a minimum thickness of 0.080 inches. Class I panels are flush mounted directly to wooden, U-2 steel, or tubular steel posts, as directed in Standard Plan S-614-2 of the *CDOT Standard Plan – M & S Standards* **(22)**.

Class II sign panels are also single-sheet aluminum with minimum thicknesses of 0.100 inches mounted on wooden, U-2 steel, or tubular steel posts, however, Class II signs are mounted with one or two aluminum backing zees as outlined in Standard Plan S-614-3 of the *CDOT Standard Plan – M & S Standards* **(22)**.

Class III signs are guide or informational signs constructed of 0.125-inch minimum thickness sheet aluminum and mounted with backing zees. Class III signs may be located either on overhead sign structures according to the Standards for Overhead Sign Structures or on the ground using wooden, tubular steel, or W-beam shaped steel posts.

20.6.4 Ground Sign Supports and Foundations (Class III)

Determining the requirements for Class III ground sign supports and foundations is the responsibility of the designer. Standard Plan S-614-6 of the *CDOT Standard Plan – M & S Standards* **(22)** provides data for determining sign supports and concrete footing sizes for all Class III ground sign installations. Class III panels may require either wooden, tubular steel, or W–beam shaped steel supports depending on the panel size and the applied moment due to wind loads. CDOT Standards use a design wind speed of 90 mph for wind loading in most locations. Breakaway sign support requirements are found in Standard Plan S-614-5 of the *CDOT Standard Plan – M & S Standards* **(22)** for both wood and steel sign supports.

Material quantities for sign supports and concrete footings are detailed on the Tabulation of Signs provided in the plans for any permanent signing project done by the Safety and Traffic Engineering Branch.

20.6.5 Overhead Sign Structures

Overhead sign structures used on the State highway system are classified into three categories:

- Sign bridges
- Cantilever sign structures
- Butterfly sign structures

The type of overhead sign structure required for a project is covered in Standard Plan S-614-50 of the *CDOT Standard Plan – M & S Standards* **(22)**, and depends on the location and the number of sign panels needed. Once the panel sizes and span lengths are known, the structural and foundational requirements of the structure are determined using the *CDOT Standard Plan – M & S Standards* **(22)** developed by Staff Bridge Branch.

Standard Plan S-614-50 of the *CDOT Standard Plan – M & S Standards* **(22)** should be included in all plans that require overhead sign structures. Plan sheets for overhead sign structures not found in the *CDOT Standard Plan – M & S Standards* **(22)**, including cantilevers and butterfly sign structures, can be obtained from the Staff Bridge Branch.

20.6.6 Cross Sections at Class III and Overhead Sign Structure Locations

Cross sections are required for Class III and larger sign installations using appropriate stationing. Cross sections should extend 50 to 100 feet beyond the edge-of-traveled way, depending on the lateral placement of the sign. All features such as curb and gutter, guardrail, ditches, fences, right of way lines, bikeways, and roadways should be indicated. Class III panels should be detailed on the cross sections and placed the appropriate lateral distance from the edge-of- traveled way. The bottom of the panel shall be located in accordance with Standard Plan S-614- 1 of the *CDOT Standard Plan – M & S Standards* **(22)**.

For sign bridge structures, a cross section from the median centerline to 41 feet beyond the edge-of-traveled way should be obtained.

20.7 SPECIFICATIONS

20.7.1 Standard Specifications

All standard specifications for traffic control devices related to construction are found in the *CDOT Standard Specifications for Road and Bridge Construction* **(32)**.

20.7.2 Standard Special Provisions

Traffic Standard Special Provisions are additions and revisions to the *CDOT Standard Specifications for Road and Bridge Construction* **(32)** initiated by Safety and Traffic Engineering and approved by the Joint CCA/CDOT Specifications Committee. These provisions are unique to a selected group of projects or are intended for temporary use. Standard Special Provisions to be used on construction projects can be accessed on the *Construction Specifications Web Page* **(38)**.

20.7.3 Traffic Project Special Provisions

Traffic Project Special Provisions are additions and revisions to the *CDOT Standard Specifications for Road and Bridge Construction* **(32)** unique to a particular project. They are available for use on a project-by-project basis and are posted on the *CDOT Safety and Traffic Engineering Web Page* **(39)**.

20.8 SIGNALS

20.8.1 Signal Plans

Traffic signals play an important role in the safe and steady flow of traffic. The *MUTCD Part 4* **(4)** provides the criteria for the design and installation of traffic signals. The traffic signal plan sheets provided by the Region Traffic Section will show the placement of the signal poles, heads, conduit, pull boxes and all other related signal equipment. Standard Plans S-614-40 and S-614-40A of the *CDOT Standard Plan – M & S Standards* **(22)** provide details of the signal equipment required by CDOT.

When designing sidewalks and channelization islands, consideration of *Americans With Disabilities Act* (ADA) standards and the needs of able-bodied pedestrians should be taken into account. Poles, boxes and other related equipment should be placed so that pedestrians have unobstructed walkways.

20.8.2 Warrant Studies

Properly designed traffic signals make intersections safer and more efficient by improving traffic flow. However, signals are not cure-alls for improving traffic flow and reducing crashes at all intersections. Traffic signals should be warranted before they can be installed. Specific criteria are given in Part 4 of the *MUTCD* **(4)** for the installation of traffic signals. Even if an intersection meets warrant criteria, careful consideration should be given to other traffic control devices before a signal is decided upon.

20.9 PAVEMENT MARKINGS

20.9.1 Permanent

Adequate pavement markings have been a cost-effective means of enhancing both traffic safety and mobility. CDOT requires centerline, edge line, auxiliary lane, crosswalk and other pavement markings on all roads under its jurisdiction. CDOT requires durable pavement markings on all mainline Interstate projects and on other selected roadways where traffic volumes are high or non-durable markings have not been cost-effective. “Durable” pavement marking materials are those materials capable of providing a longer service life than conventional paint.

General guidelines for the selection of pavement marking materials for roadway projects may be found in an agreement between CDOT and the FHWA titled, *CDOT Recommended Pavement Marking Practices* **(29).** Copies of this guideline are available from the Safety and Traffic Engineering Section.

Other considerations in the selection process may include the desire to use materials that are lead-free, materials that contain lower levels of volatile organic compounds (VOC’s), or materials that do both. The *MUTCD* **(4)** and Standard Plan S-627-1 of the *CDOT Standard Plan – M & S Standards* **(22)** outline the details and requirements for the proper selection and installation of all pavement markings. Refer to Section 627 of the *CDOT Standard Specifications for Road and Bridge Construction* **(32)**.

Tabulation of Pavement Marking quantities will be included in the plan sheets provided by the designers and reviewed by the Region Traffic Engineering Section for most construction projects.

20.9.2 Temporary

See section 20.5.3.

20.10 RESEARCH

The Traffic and Safety Engineering Branch in cooperation with the Division of Transportation Development Research Branch and the Materials and Geotechnical Branch continually evaluates new traffic engineering products available from private industry. For information regarding research projects on State highways, contact the Traffic and Safety Engineering Branch or the Region Traffic Engineer.

REFERENCES

1. CDOT. *CDOT Project Development Manual*, Colorado Department of Transportation, 2013 (with revisions through 2016).

2. AASHTO. *Highway Safety Design and Operations Guide,* American Association of State Highway and Transportation Officials (AASHTO), Washington, D.C., 1997.

3. AASHTO. *Roadside Design Guide*, American Association of State Highway and Transportation Officials (AASHTO), Washington, D.C., 2011.

4. U.S. Department of Transportation, Federal Highway Administration, *Manual on Uniform Traffic Control Devices for Streets and Highways (MUTCD),* Washington, D.C., 2009.

5. AASHTO. *Highway-Rail Crossing Elimination and Consolidation*, American Association of State Highway and Transportation Officials (AASHTO), publication No. HRC-1 March 1995.

6. FHWA. *Railroad-Highway Grade Crossing Handbook*, Federal Highway Administration, Washington, D.C., August 2007.

7. FHWA. *Guidance on Traffic Control Devices at Highway-Rail Grade Crossings*, FHWA Highway/Rail Grade Crossing Technical Working Group, Federal Highway Administration, November 2002.

8. CDOT. *CDOT Railroad Manual*, 2017

9. AASHTO. *A Policy of Geometric Design of Highways and Streets*, American Association of State Highway and Transportation Officials (AASHTO), Washington, D.C., 2011

10. FHWA. *Roundabouts: An Informational Guide,* Federal Highway Administration, Publication No. FHWA-RD-00-067, National Technical Information Service, McLean, VA, 2000.

11. TRB. *Highway Capacity Manual*, Transportation Research Board, National Research Council, Washington, D.C., 2010.

12. NCHRP. *NCHRP Report 480, A Guide to Best Practices for Achieving Context Sensitive Solutions (CSS),* Transportation Research Board, National Research Council, National Academy Press, Washington, D.C., 2003.

13. FHWA. *Flexibility in Highway Design*, Federal Highway Administration Publication No. FHWA-PD-97-062.

14. NCHRP. *NCHRP Report 374, Effect of Highway Standards on Highway Safety,* Transportation Research Board, National Research Council, National Academy Press, Washington, D.C., 1995.

15. NCHRP. *NCHRP Report 430, Improved Safety Information to Support Highway Design*, Transportation Research Board, National Research Council, National Academy Press, Washington, D.C., 1999.

16. NCHRP. *NCHRP Report 362, Roadway Widths for Low Traffic Volume Roads,* Transportation Research Board, National Research Council, National Academy Press, Washington, D.C., 1994.

17. FHWA. Federal Highway Administration *Interactive Highway Safety Design Model* [https://www.fhwa.dot.gov/research/tfhrc/projects/safety/comprehensive/ihsdm]

18. FHWA. *Prediction of the Expected Safety Performance of Rural Two-Lane Highways,* Federal Highway Administration Publication No. FHWA-RD-99-207, National Technical Information Service, McLean, VA, 2000.

19. CDOT Chief Engineer's Policy Memo 26, *Context Sensitive Solutions (CSS) Vision for CDOT* , 2005: [http://foreclosure.co.clear-creek.co.us/Visitor/Signed_CSS_Policy_memo26.pdf]

20. NCHRP. *NCHRP Report 350, Recommended Procedures for the Safety Performance Evaluation of Highway Features*, Ross, Sicking, Zimmer, and Michie, Transportation Research Board, National Research Council, National Academy Press, Washington, D.C., 1993.

21. CDOT. Policy Directive 902.0 Shoulder Policy, Colorado Department of Transportation, June 17, 1999.

22. CDOT. *CDOT Standard Plans – M & S Standards*, Colorado Department of Transportation, 2012

23. AASHTO. *Guide for the Development of Bicycle Facilities*, American Association of State Highway and Transportation Officials (AASHTO), Washington, D.C., 2012.

24. FHWA Rumble Strip Web Page: [https://safety.fhwa.dot.gov/roadway_dept/pavement/rumble_strips]

25. NCHRP. *NCHRP Synthesis 191*, *Use of Rumble Strips to Enhance Safety*, Harwood, Transportation Research Board, National Research Council, National Academy Press, Washington, D.C., 1993

26. AASHTO/NCHRP. *NCHRP PROJECT 17 – 18, Implementing the AASHTO Strategic Highway Safety Plan*; American Association of State Highway and Transportation Officials (AASHTO), Washington, D.C., 2009

27. CDOT. *Crash Cushion and End Treatment Selection Guide*. Maintained by the CDOT Standards Engineer and available on CDOT's Design Support web page: [https://www.codot.gov/business/designsupport/design-docs/safety-selection-guide/list/view]

28. CDOT. *Colorado Supplement to the Standard Highways Signs*. Colorado Department of Transportation, 2012.

29. CDOT. *CDOT Recommended Pavement Marking Practices*, CDOT Colorado Department of Transportation Safety and Traffic Engineering Branch, Revised September 1998.

30. FHWA. U.S. Department of Transportation, Federal Highway Administration. *2012 Supplement to Standard Highway Signs2004 Edition*, U.S. Department of Transportation, Federal Highway Administration. Washington, D.C., 2012.

31. FHWA. U.S. Department of Transportation, Federal Highway Administration. *Standard Highway Signs*, U.S. Department of Transportation, Federal Highway Administration. Washington, D.C., 2004.

32. CDOT. *Standard Specifications for Road and Bridge Construction*, Colorado Department of Transportation, 2017.

33. AASHTO. *Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals*, Washington, D.C.: AASHTO, 2015.

34. CDOT. *Colorado Supplement to the MUTCD*. Colorado Department of Transportation, latest edition.

35. ITE. *Traffic Control Devices Handbook*, Institute of Transportation Engineers, Washington, D.C. 2013.

36. ITE. *Traffic Engineering Handbook*. Six Edition. Wolfgang S. Homburger, (Ed.). Institute of Transportation Engineers, Prentice Hall, Englewood Cliffs, N.J., 2009.

37. ITE. *Transportation Planning Handbook*. Third Edition. Wolfgang S. Homburger, (Ed.). Institute of Transportation Engineers, Prentice Hall, Englewood Cliffs, N.J., 2009.

38. CDOT. Construction Specifications Web Page, Colorado Department of Transportation: [https://www.codot.gov/business/designsupport/cdot-construction-specifications]

39. CDOT. Traffic Special Provisions, CDOT Safety and Traffic Engineering Web Page, Colorado Department of Transportation: [https://www.codot.gov/library/traffic/traffic-specifications]

40. AASHTO. *Manual for Assessing Safety Hardware (MASH)*, American Association of State Highway and Transportation Officials (AASHTO), Washington, D.C., 2016

A

B

C **Page**

D

E