

## 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 established current Colorado Department of Transportation (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.

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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 a team of CDOT engineers from project delivery representatives from the all CDOT regions and the Project Development Branch of Staff Services, TSM&O and the Division of Transportation Development (DTD).

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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 good engineering judgment, along with the PGDHS named above. 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. Exceptions-Variations from these guidelines minimum design criteria should be thoroughly evaluated and documented in accordance with CDOT's design exception policies.

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The standards presented in this guide represent required values. These values imply the lowest acceptable limit in design. Design policies and standards are not inflexible. Higher standards may be used within reasonable economic limits. To ensure uniform practice on a statewide basis, lower design standards will not be used without a Design Exception Variance Request or a design decision, as outlined in the CDOT Project Development Manual.

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

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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*.

2018

Comments on this Guide may be sent to:

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Standards and Specifications Unit  
2829 W. Howard Place  
Denver, Colorado 80204

Or ~~you may call 303-757-9474~~ the design program manager.

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

Introduction

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**CHAPTER 1**

**FUNCTIONAL AND CONTEXTUAL HIGHWAY CLASSIFICATIONS**

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## CHAPTER 1 FUNCTIONAL AND CONTEXTUAL HIGHWAY CLASSIFICATIONS

### 1.0 INTRODUCTION

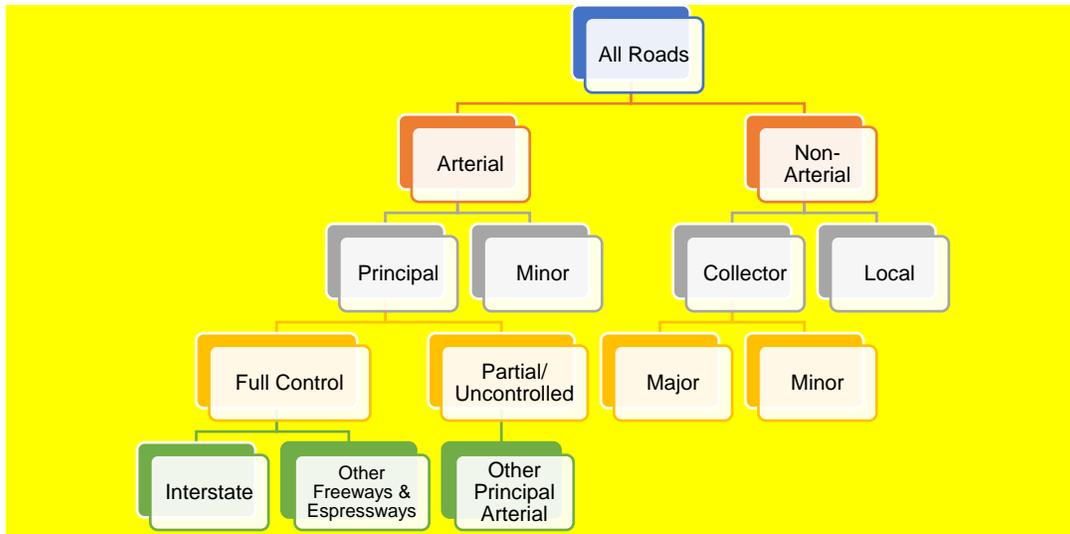
The function of a highway is to provide a facility that allows the movement of people, goods, and services. Different types of facilities are needed for various vehicle travel movements. To differentiate between the types of highways, functional and contextual classifications have been developed so that 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. Travel lanes are typically separated by a physical barrier, aBy definition, Freeways are characterized by full access control with access points are limited to on/off ramps and no at grade intersections. Expressways are more common in rural settings where, and at grade intersections are limited permitted to varying degrees depending on context. Like interstates, In general, these types of

- roadways favor-maximize 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 ~~with other roadways~~. 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 through 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 context is relative to urban context.~~
- 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 institutional 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 ~~should~~ can be ~~provided~~ used to evaluate the feasibility of the design variance.

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1. CDOT. *CDOT Online Transportation Information System (OTIS)*
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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](https://www.fhwa.dot.gov/planning/processes/statewide/related/highway_functional_classifications/fcauab.pdf)
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## 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.

<b>Level of Service</b>	<b>General Operating Conditions</b>
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**

<b>Functional Class</b>	<b>Appropriate Level of Service for Specified Combinations of Area and Terrain Type</b>			
	<b>Rural Level</b>	<b>Rural Rolling</b>	<b>Rural Mountainous</b>	<b>Urban and Suburban</b>
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)*:  
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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.

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

**Table 3-1 Sight Distance**

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

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:

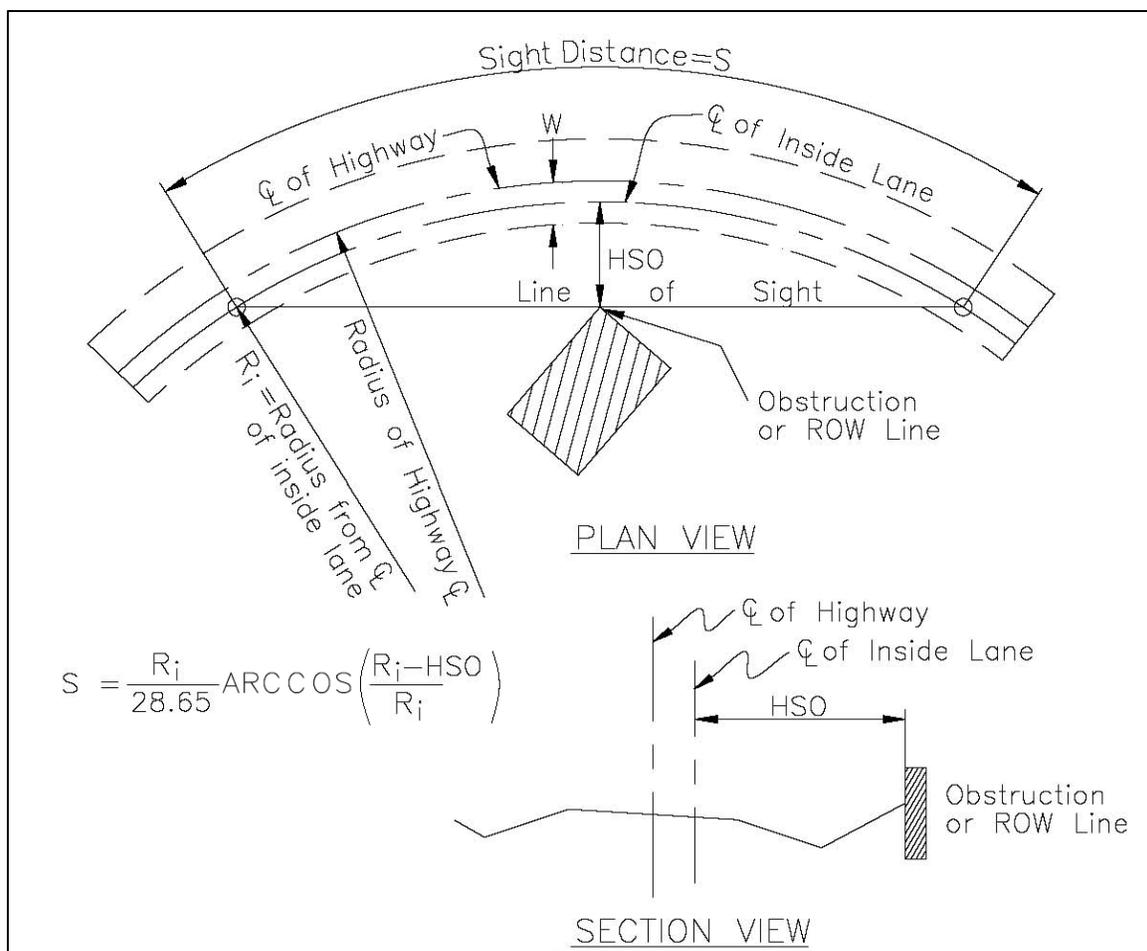
- Interchange and intersection locations where unusual or unexpected maneuvers are required.
- Changes in cross section such as toll plazas and lane drops.
- Areas of concentrated demand where there is apt to be "visual noise" whenever sources of information compete, as those from roadway elements, traffic,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

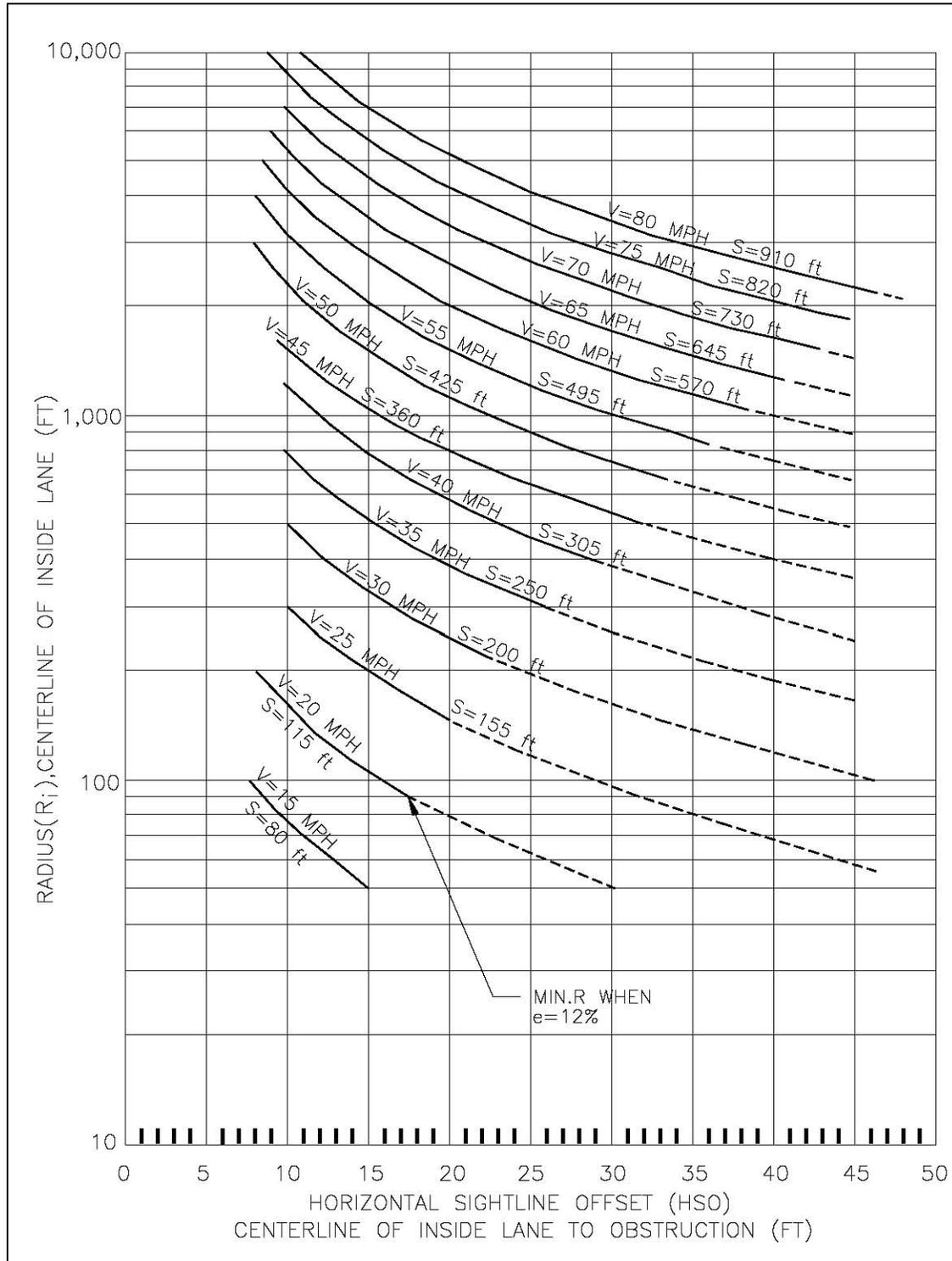
Where:

$R_i$  = 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.

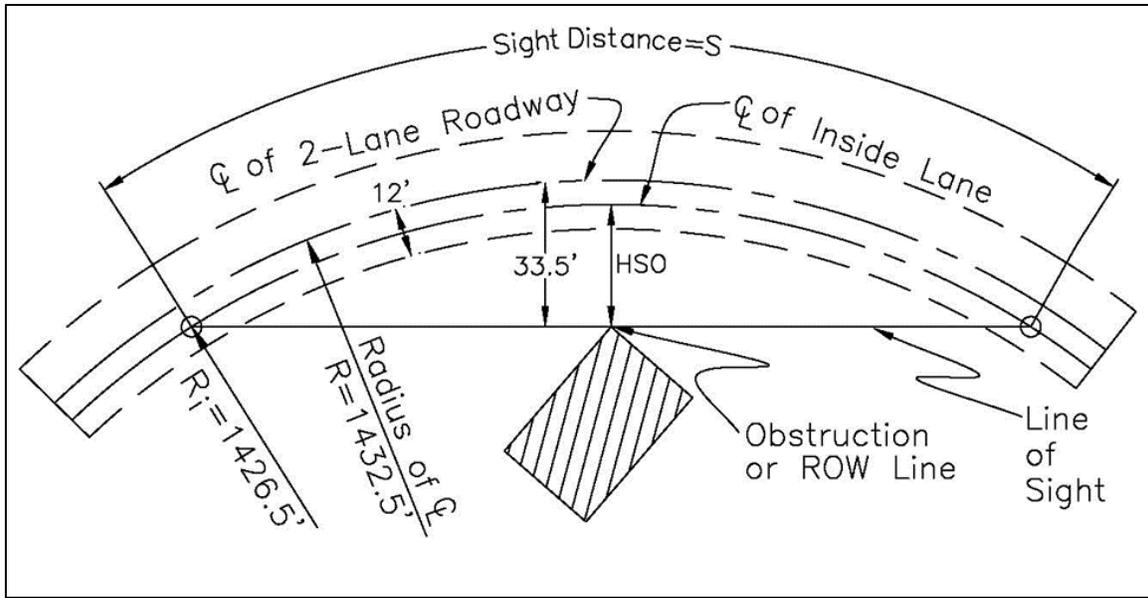
S = available stopping sight **distance** (feet)

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.

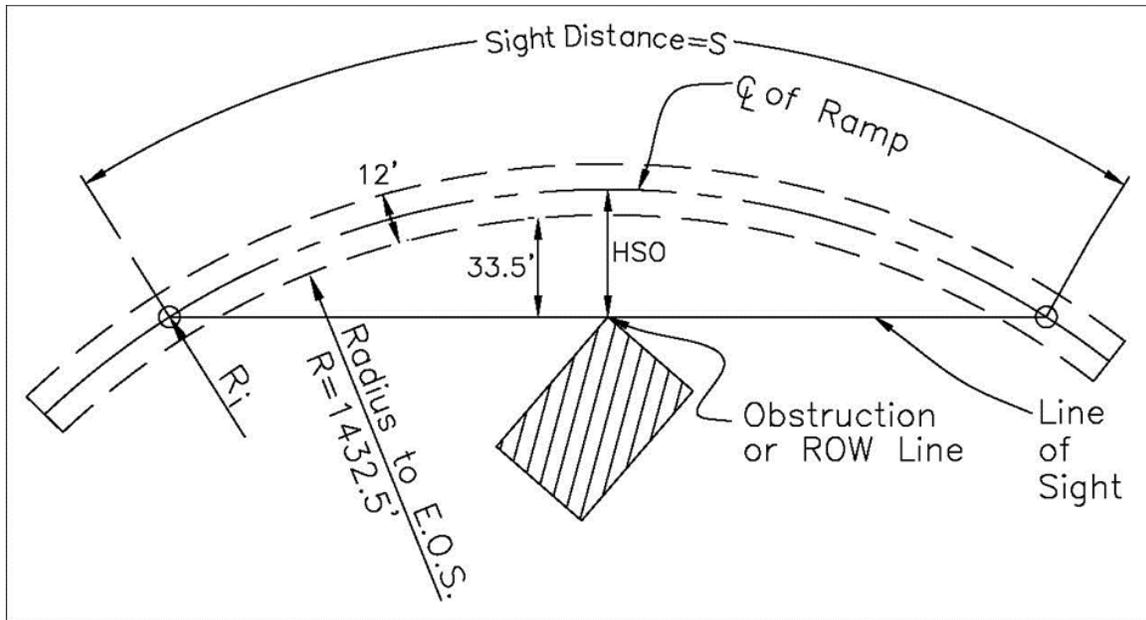


**Figure 3-3 Example of Horizontal Stopping Sight Distance on a Two-Lane Roadway**

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

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

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

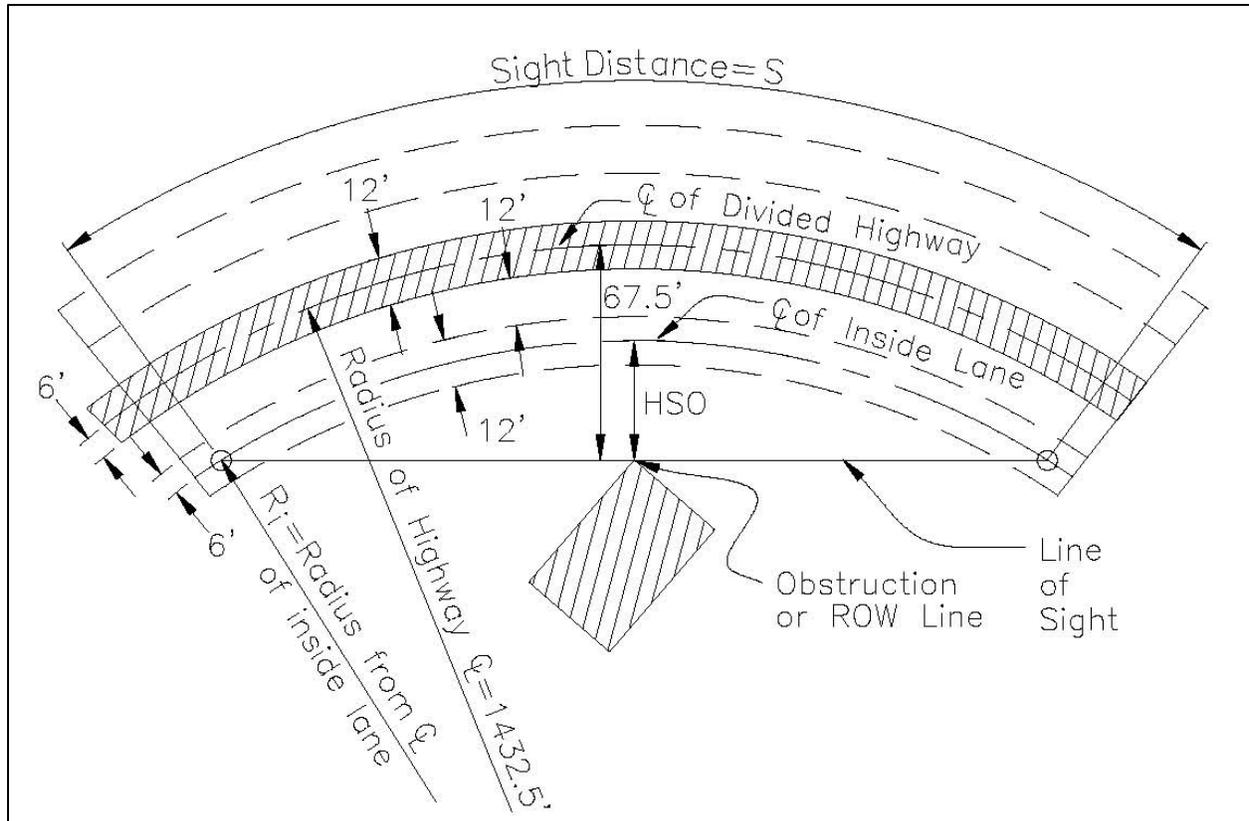


**Figure 3-4 Example of Horizontal Stopping Sight Distance on a Ramp**

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

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

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



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

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

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

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

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  $L$ ,

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

When  $S$  is greater than  $L$ ,

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

Where:

$L$  = length of vertical curve, in feet

$A$  = algebraic difference in grades, in percent

$S$  = Sight distance, in feet

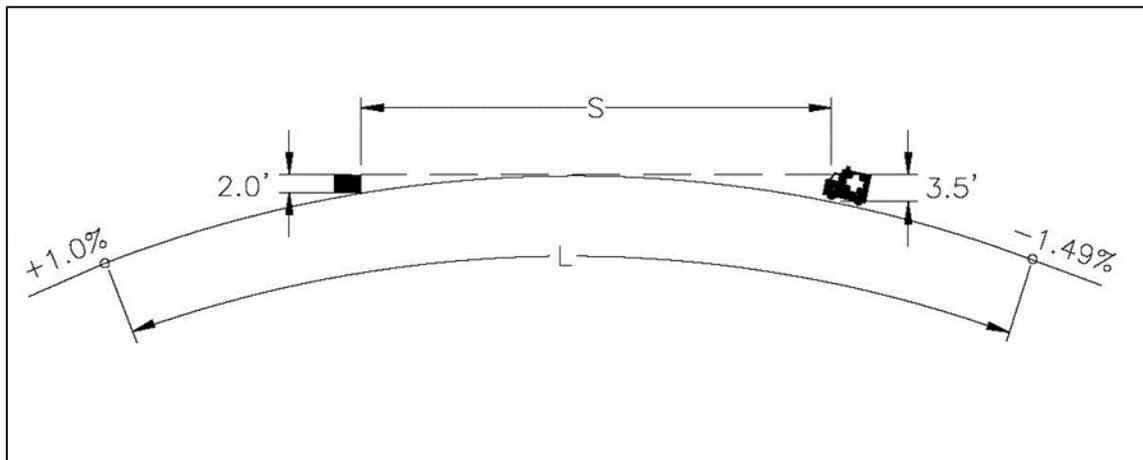


Figure 3-6 Example of Crest Vertical Curve

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):

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

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

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

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

### 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  $L$ ,

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

When  $S$  is greater than  $L$ ,

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

Where:

$L$  = length of sag vertical curve, in feet

$S$  = light beam distance, in feet

$A$  = algebraic difference in grades, in percent

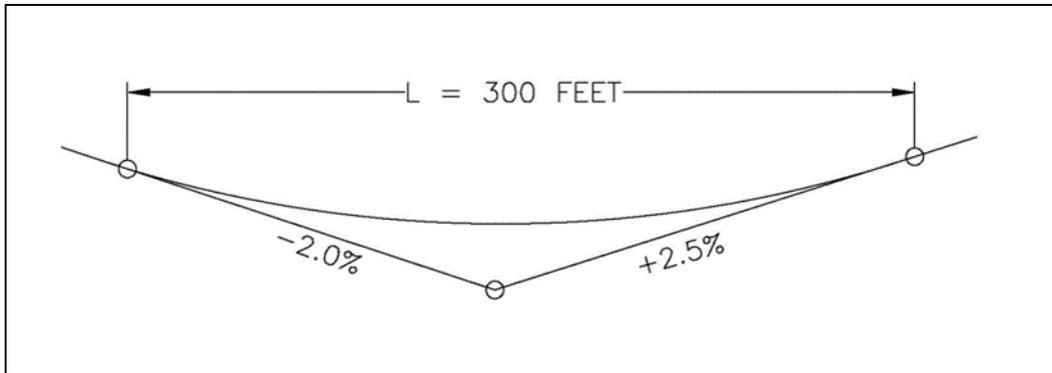


Figure 3-7 Example of Sag Vertical Curve

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  $K(\text{Sag})$  value of 66.7,  $V = 40.9$  mph.

$\therefore$  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):

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

### 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  $L$ ,

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

When  $S$  is greater than  $L$ ,

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

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 8-7.6 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.

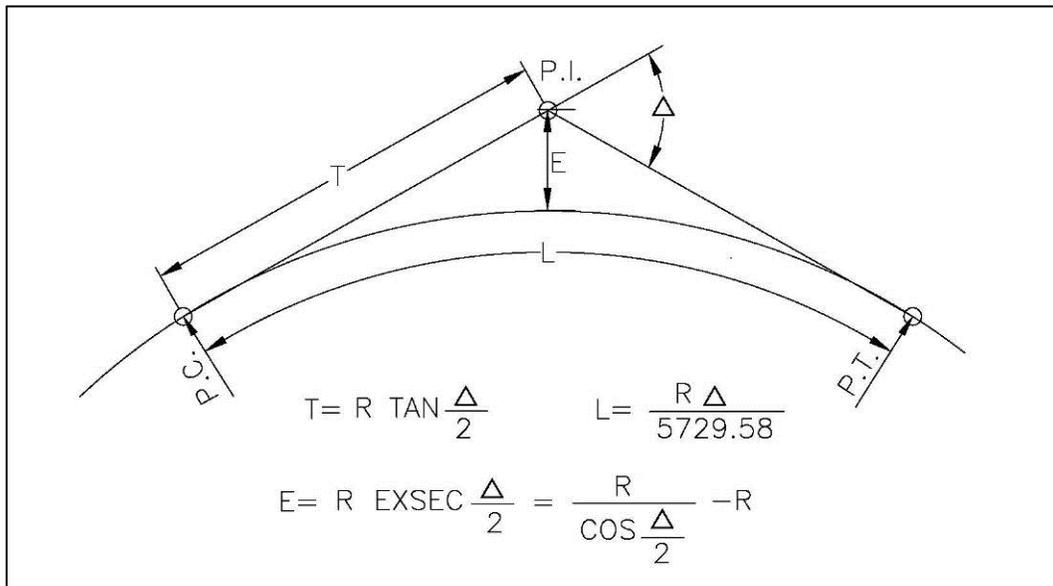
Every effort should be made to exceed the minimum curve radii. Minimum curve radii should be used only when the cost of realizing a higher standard is not consistent with the benefits. The final considerations for the safety of any curve should be the combination of the factors of radius, sight distance,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

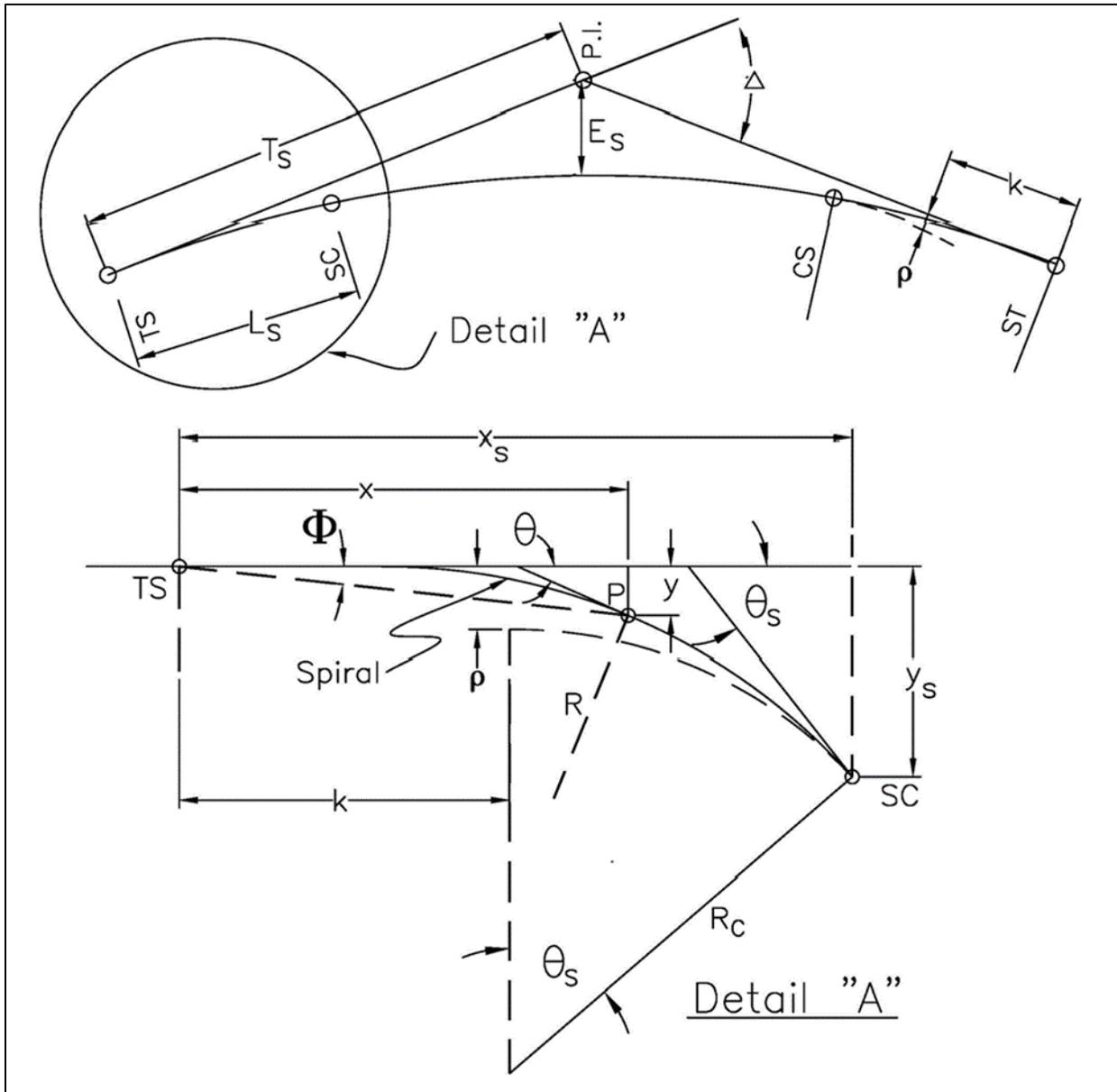
$\Delta$  = deflection angle between the tangents, decimal degrees

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:

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

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

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

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

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

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

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

Where L is in feet and  $\theta$  is measured in decimal degrees.

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

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

Where  $\theta$  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

$\theta$  = central angle of spiral arc L

$\theta_s$  = central angle of spiral arc  $L_s$ , called "spiral angle"

$\Phi$  = 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

$\Delta$  = total central angle of the circular curve

$\Delta_c$  = 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

$\rho$  = 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)

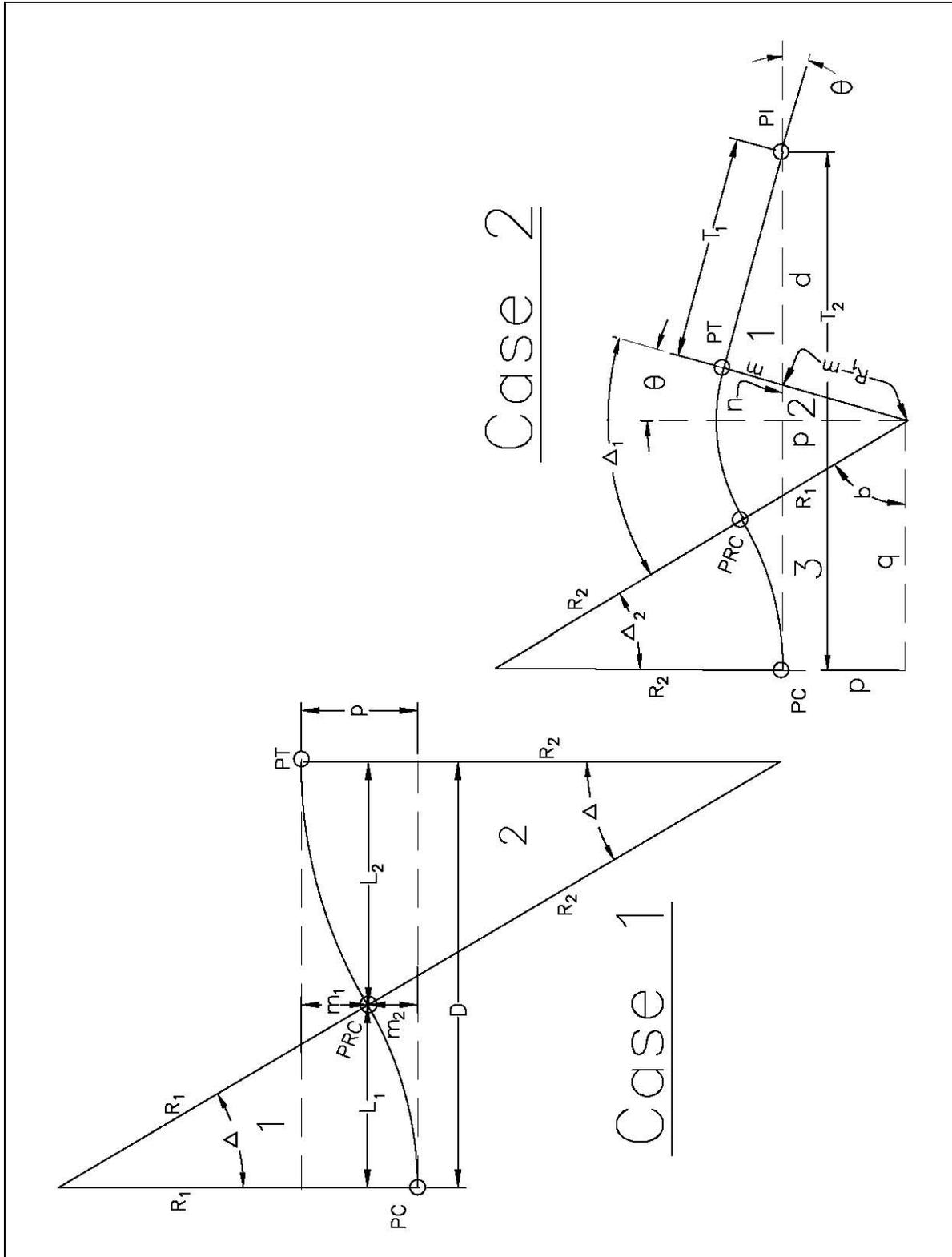


Figure 3-10 Reverse Curves

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,

$$p = m_2 + m_1$$

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

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

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

And,

$$D = L_1 + L_2 \quad [3-18]$$

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

$$D = (R_1 + R_2) \sin \Delta \quad [3-19]$$

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  $\theta$ , 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 ( $\Delta_1$  &  $\Delta_2$ ) 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  $R_1 - m$ , 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  $R_1 + R_2$ , from which the angles  $\Delta_2$  and  $b$  and the distance  $q$  can be found.

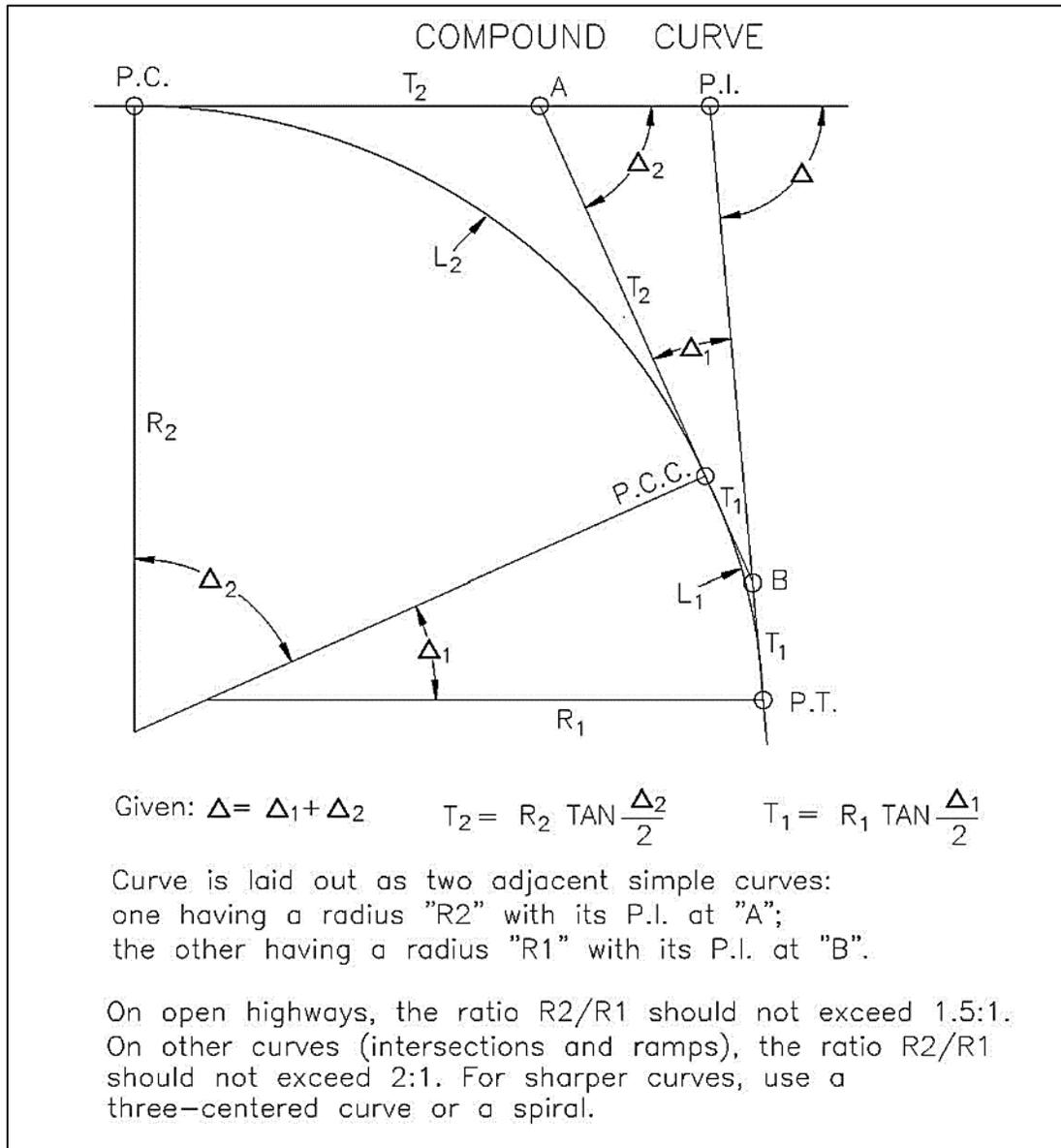
$$\text{Then} \quad \Delta_1 = \theta + \Delta_2 \quad [3-20]$$

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

### 3.2.2.4 Compound Curves

Compound circular curves are ~~Two-two~~ 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.



**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 minimum 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 to or 8 percent after the designer due to snow and ice concerns 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 6 percent should be the maximum superelevation rate of 6 percent is typically chosen in considered in urban areas. The selection of 6 percent as the maximum superelevation rate is also common as well as 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.

~~As mentioned in~~ Refer to section 3.2.3.2, a 6 percent maximum superelevation rate is for the recommended maximum superelevation rates for the design of 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.

<i>e</i> (%)	<b>R (ft) for Design Speed (mph)</b>						
	<b>15</b>	<b>20</b>	<b>25</b>	<b>30</b>	<b>35</b>	<b>40</b>	<b>45</b>
-2.0	50	107	198	333	5210	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.

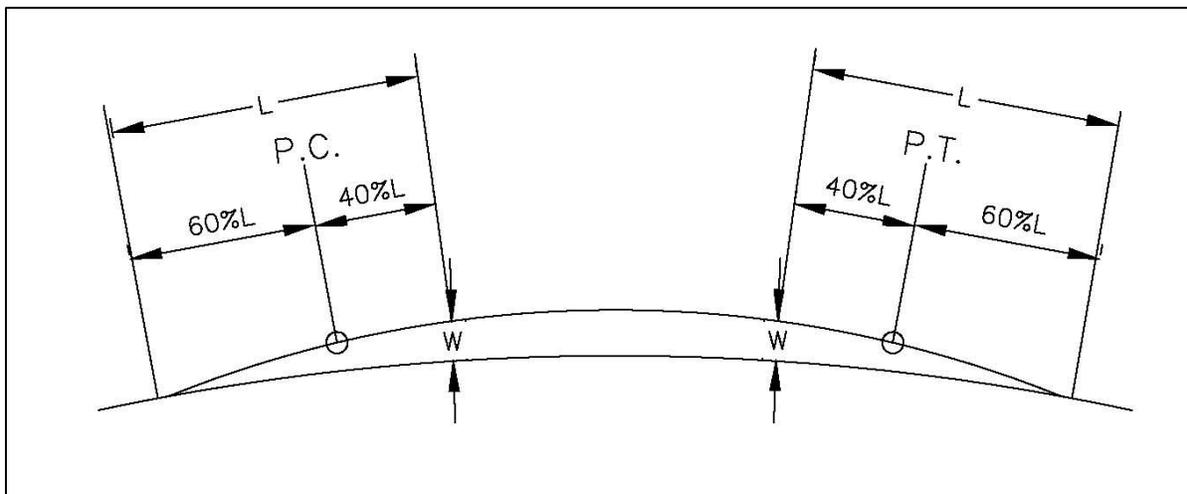
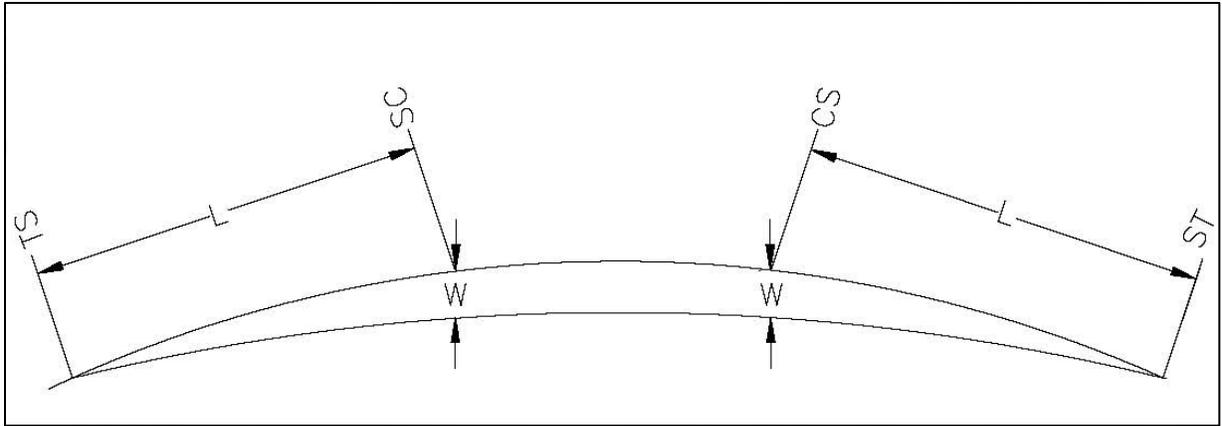


Figure 3-12 Widening on a Simple Curve



**Figure 3-13 Widening on a Spiral Curve**

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

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

$$\text{For } V > 40 \text{ mph} \quad L \geq WV \quad [3-23]$$

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

$$\text{Since } V \leq 40, \quad \frac{WV^2}{60} = \frac{3.5(30)^2}{60} = 52.5 \text{ ft}$$

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

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.

	HIGHWAY UNDERPASSES		RAILWAY UNDERPASSES ***		OVERHEAD WIRES	
	HORIZONTAL	VERTICAL*	HORIZONTAL	VERTICAL* ‡	HORIZONTAL▲	VERTICAL*
Local Rural Roads	<b>E</b>	14.5 feet	<b>F</b>	<b>G</b>	<b>A</b>	<b>H</b>
Local Urban Streets**					<b>B</b>	
Rural Collectors					<b>C</b>	
Urban Collectors**					<b>D</b>	
Rural Arterials						
Urban Arterials**						
Freeways		16.5 feet #				

**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 <sup>a</sup> [Table 8-1 (1)]										
LEVEL					4 <sup>c</sup>	4 <sup>c</sup>	3 <sup>c</sup>	3	3	3	3
ROLLING					5 <sup>c</sup>	5 <sup>c</sup>	4 <sup>c</sup>	4	4	4	4
MOUNTAINOUS					6	6	6	5	5	--	--
	RURAL ARTERIALS [Table 7-2 (1)]										
LEVEL			5 <sup>c</sup>	5 <sup>c</sup>	4 <sup>c</sup>	4 <sup>c</sup>	3	3	3	3	3
ROLLING			6 <sup>c</sup>	6 <sup>c</sup>	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 <sup>b</sup> [Table 6-2 (1)]										
LEVEL	7 <sup>c</sup>	7 <sup>c</sup>	7	7	6	6	5				
ROLLING	10 <sup>c</sup>	9	8	8	7	7	6				
MOUNTAINOUS	12	10	10	10	9	9	8				
	URBAN COLLECTORS <sup>b</sup> [Table 6-8 (1)]										
LEVEL	9 <sup>c</sup>	9	9	8	7	7	6				
ROLLING	12 <sup>c</sup>	11	10	9	8	8	7				
MOUNTAINOUS	14 <sup>c</sup>	12	12	11	10	10	9				
	LOCAL RURAL ROADS [Table 5-2 (1)]										
LEVEL	8 <sup>d</sup>	7	7	7	6	6	5				
ROLLING	11	10	10	9	8	7	6				
MOUNTAINOUS	16	14	13	12	10	10	--				
<sup>a</sup> Grades one percent steeper than the value shown may be used in urban areas.											
<sup>b</sup> 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.											
<sup>c</sup> Design speed shown not recommended (less than minimum).											
<sup>d</sup> 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.



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 and that should be nearly 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. is a consideration for vertical curves. The effect of change is greater on sag than on crest vertical curves. The length of a vertical curve needed to satisfy the comfort level is about half that needed to satisfy the headlight sight distance.

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.

$$\text{For } S \geq 45 \text{ mph,} \quad L = WS \quad [3-24]$$

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

Where:

L = Length of taper, ft

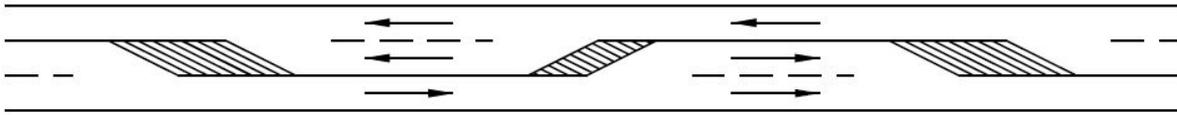
W = Width, ft

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  $\leq 45$  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.

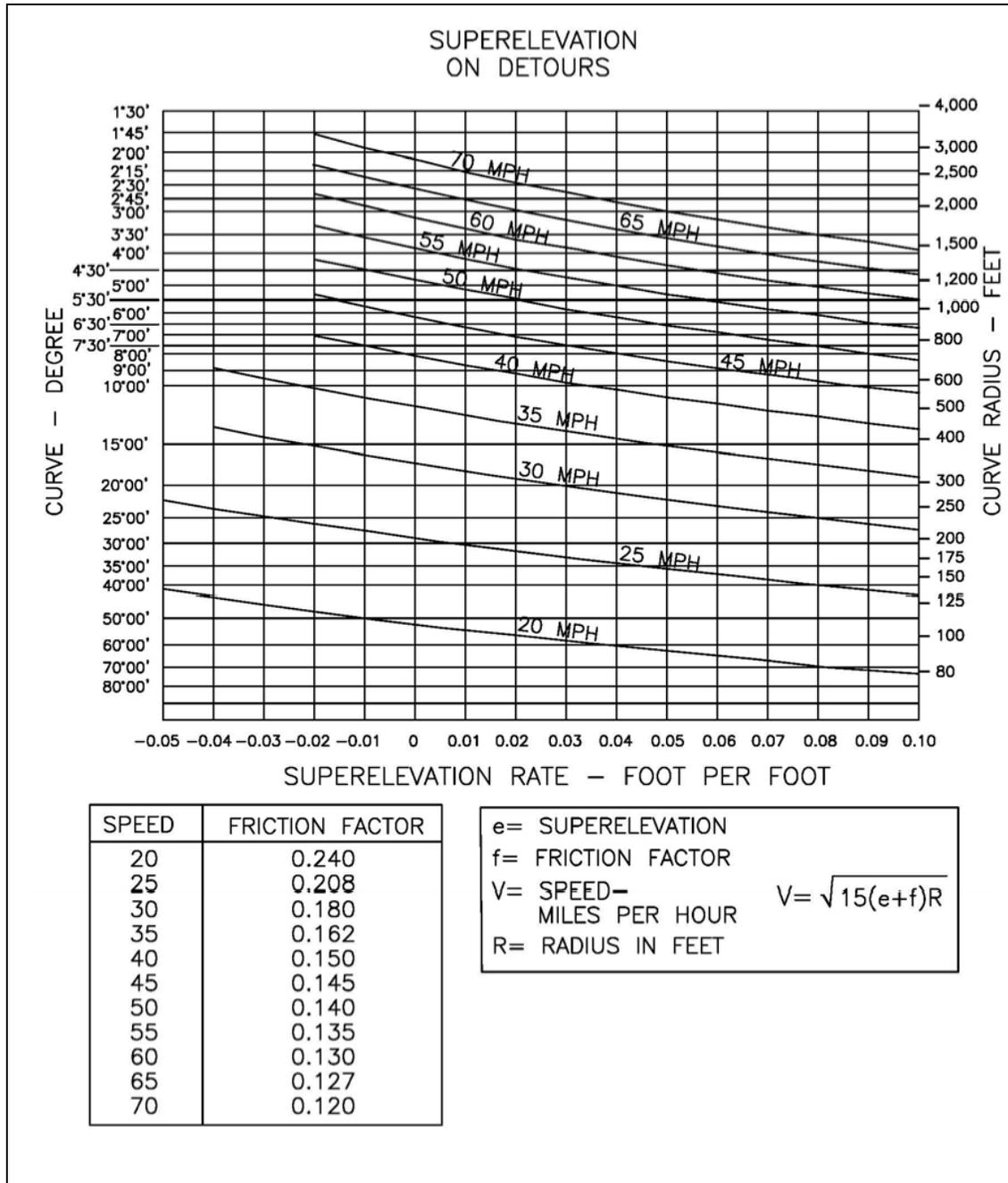
### **3.5.6 Detour Superelevation**

**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.



**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.

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## **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.

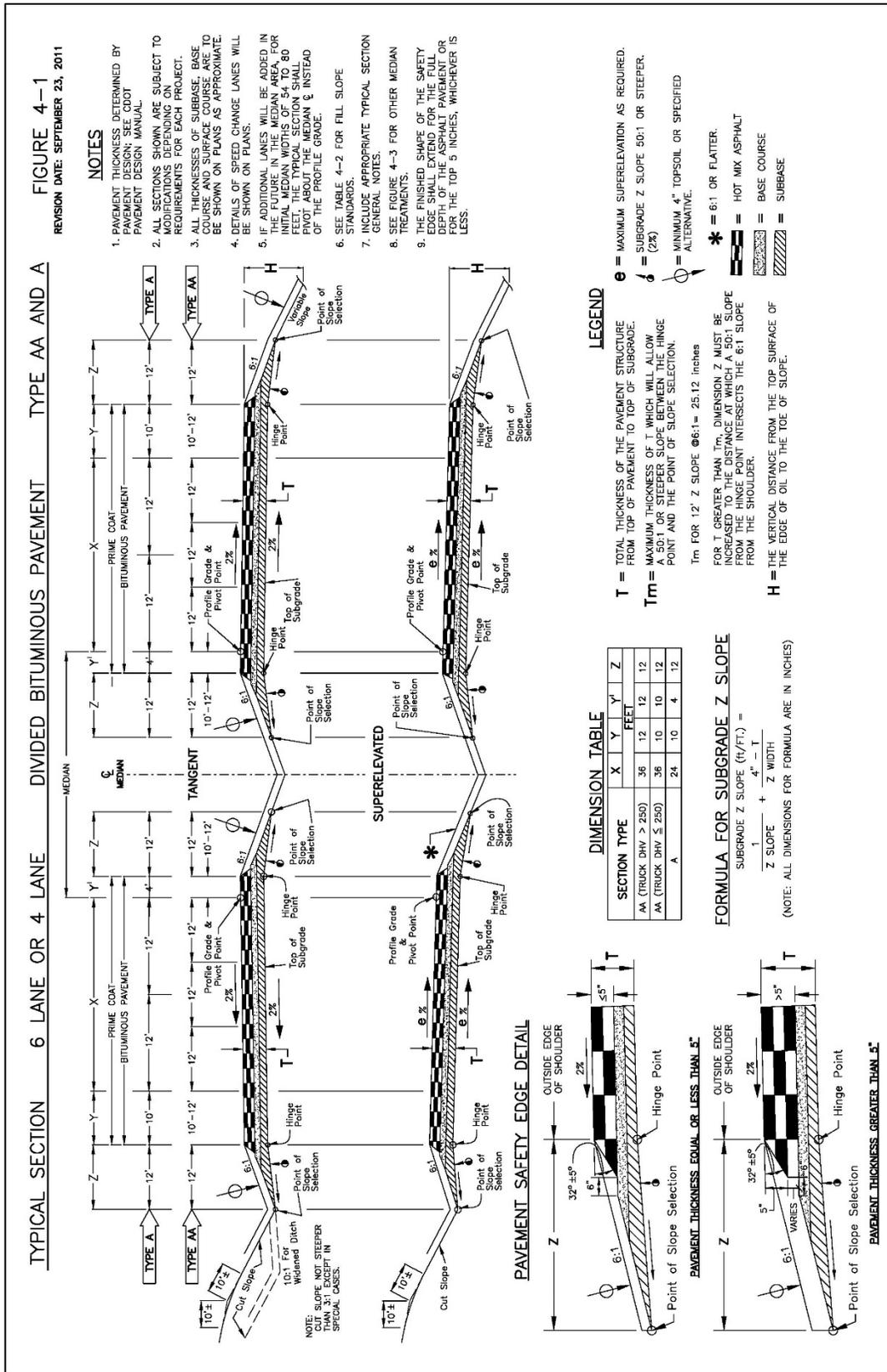
- Profile Grade: The point on the pavement surface, defined by its location on the vertical alignment of the roadway, from which all other points and slopes on the cross section are determined.
- Pivot Point: The point on the pavement surface about which the cross slope of the roadway is rotated to effect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

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

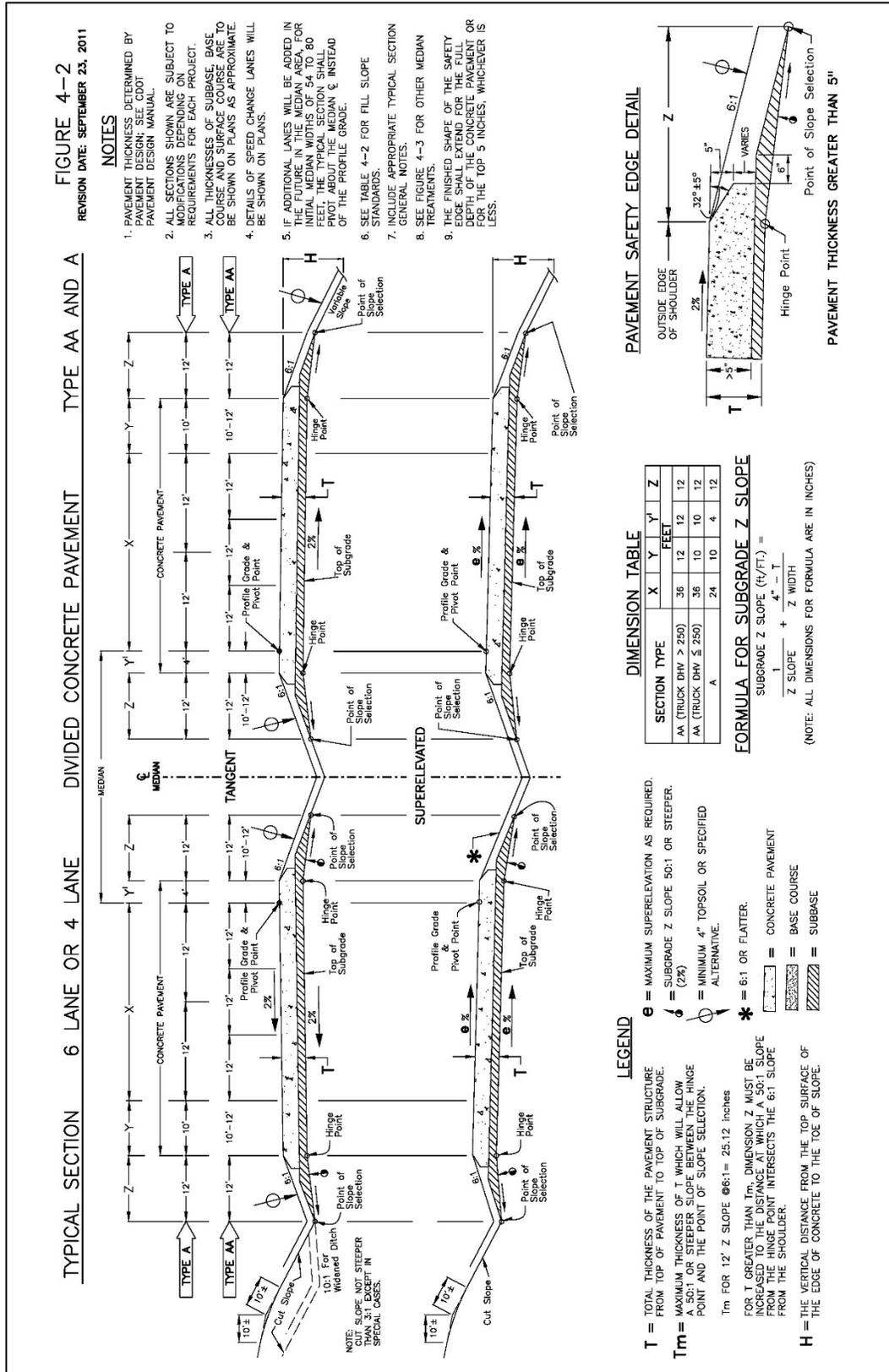
1. "Types" refers to details shown on Figures 4-1 through 4-5.
2. Shoulder widths may not apply when roadway has curb and gutter, speed-change lanes, etc.
3. See *Highway Capacity Manual* (4).
4. Where the DDHV for truck traffic exceeds 250 veh/h, a paved shoulder width of 12 feet should be considered.
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**

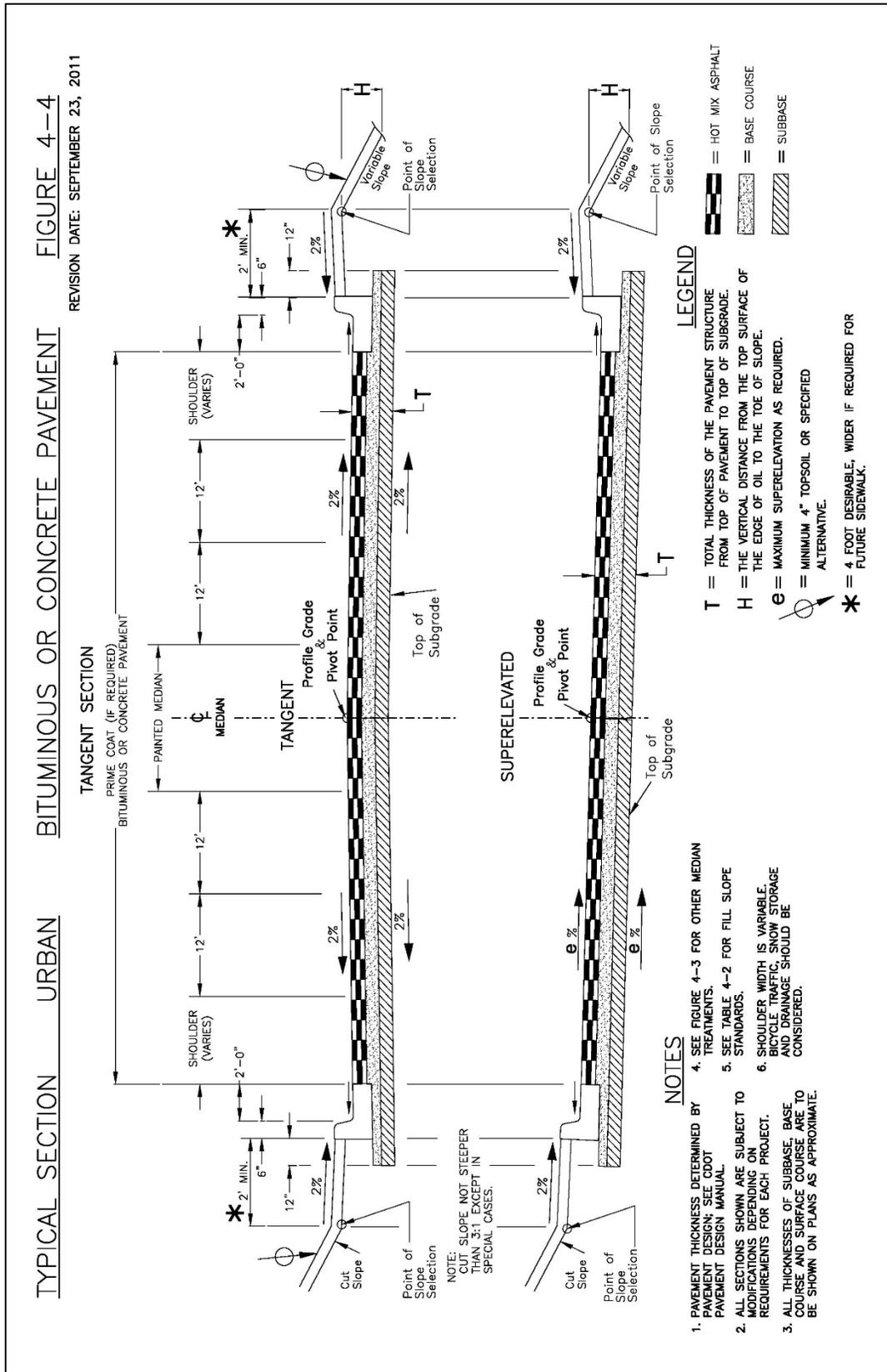


**Figure 4-1 Typical Sections for Divided Bituminous Pavement**

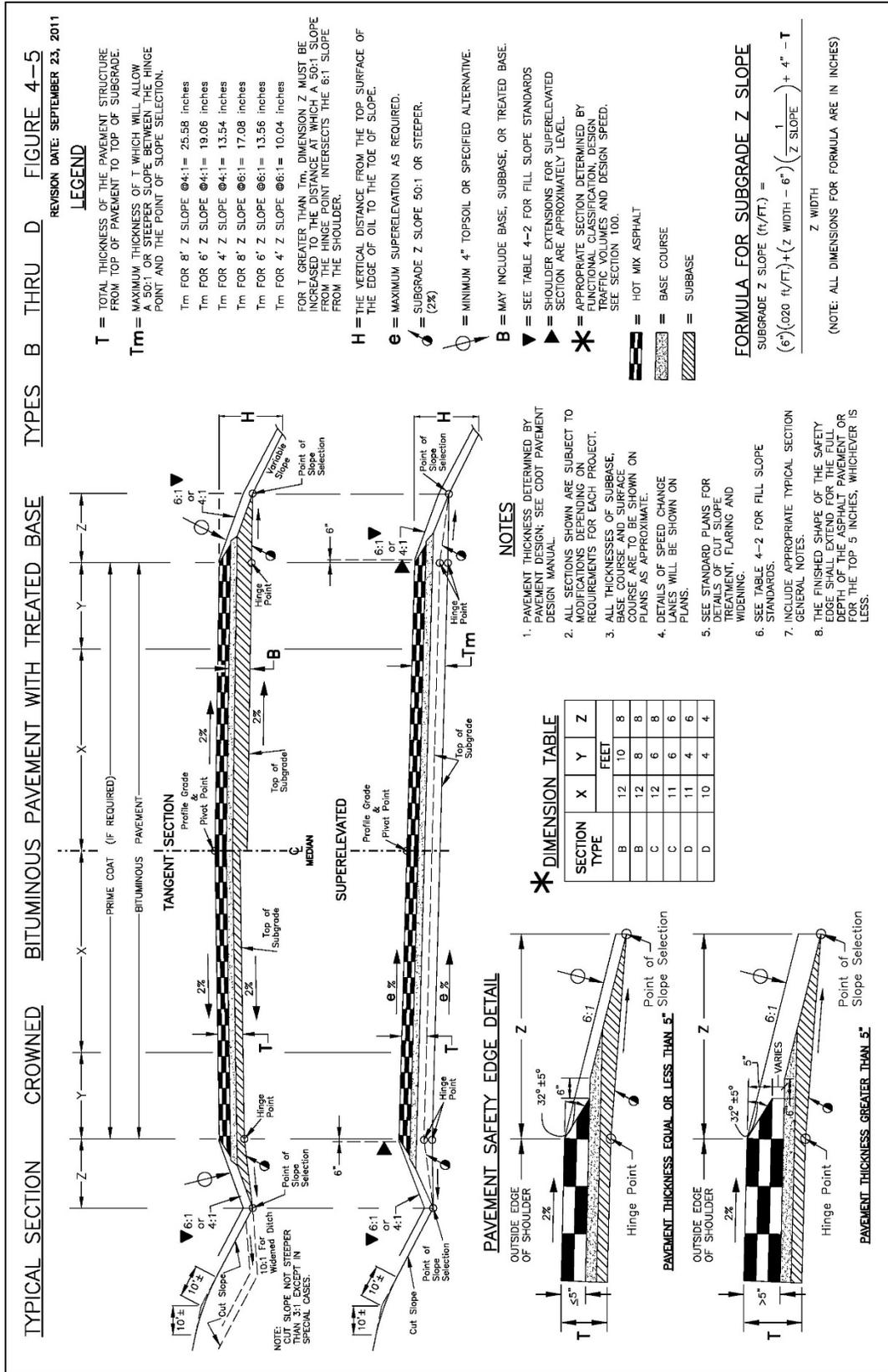


**Figure 4-2 Typical Sections for Divided Concrete Pavement**





**Figure 4-4 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 3:1 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 3:1 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'	Z, then 6:1	Z, then 4:1
	> 4' to 10'	Z, then 4:1	Z, then 4:1
	> 10' to 15'	Z, then 4:1	Z, then 3:1
	> 15'	Z, then 3:1	Z, then 3:1
‡2 lane (Z=8' @ 6:1 or 6' @6:1 or 4' @ 6:1)	≤ 4'	Z, then 6:1	Z, then 4:1
	> 4' to 10'	Z, then 4:1	Z, then 4:1
	> 10' to 15'	Z, then 4:1	Z, then 3:1
	> 15'	Z, then 2:1	Z, then 2:1
<p>* 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.</p>			

**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.

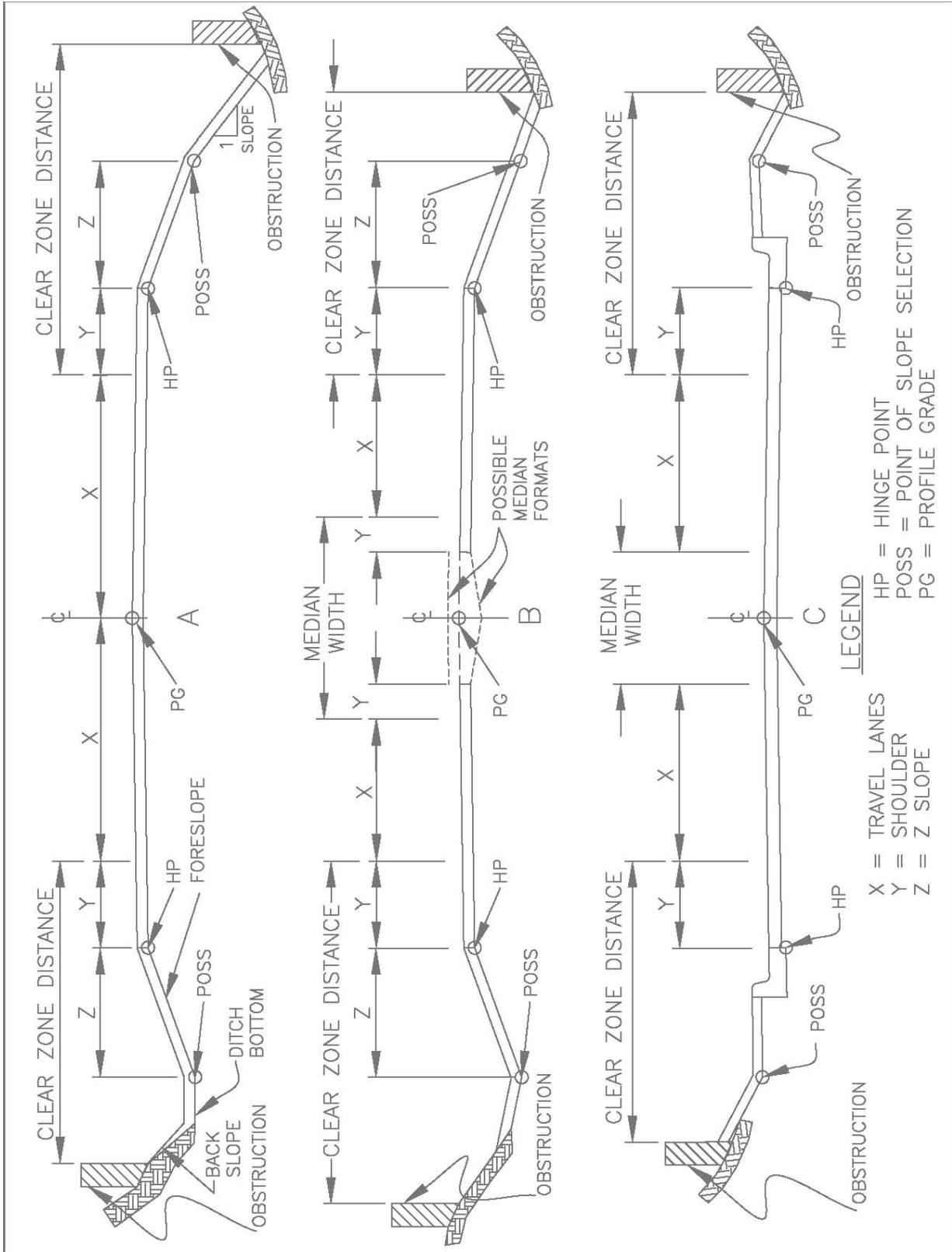


Figure 4-6 General Cross Sectional Information

## 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 sight 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 ~~needs to~~ 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.

~~CDOT's policy requires sidewalk curb ramps to be upgraded on resurfacing projects where the overlay is 1.5 inches or more in depth and on projects that are more extensive than resurfacing. See also Chapter 12 and the PROWAG (14) for details and exceptions.~~

## 4.184.17 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.

## 4.194.18 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.

### 4.19.14.18.1 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).

### 4.19.24.18.2 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).

### **4.19.34.18.3 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)*.

### **4.204.19 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.

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## 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
<b>Surface Types</b>	
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. Curb cut ramps shall be provided at crosswalks to accommodate persons with disabilities. Refer to the *CDOT Standard Plans – M & S Standards (5)*. Also see Chapter 12 of this Guide.~~

### **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**

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## 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

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## **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) <sup>a</sup> 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) <sup>b</sup>			
	4	6	6	8
<sup>a</sup> On roadways to be reconstructed, an existing 22-ft traveled way may be retained where alignment and safety records are satisfactory.				
<sup>b</sup> 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 will 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.

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## 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

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## 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.

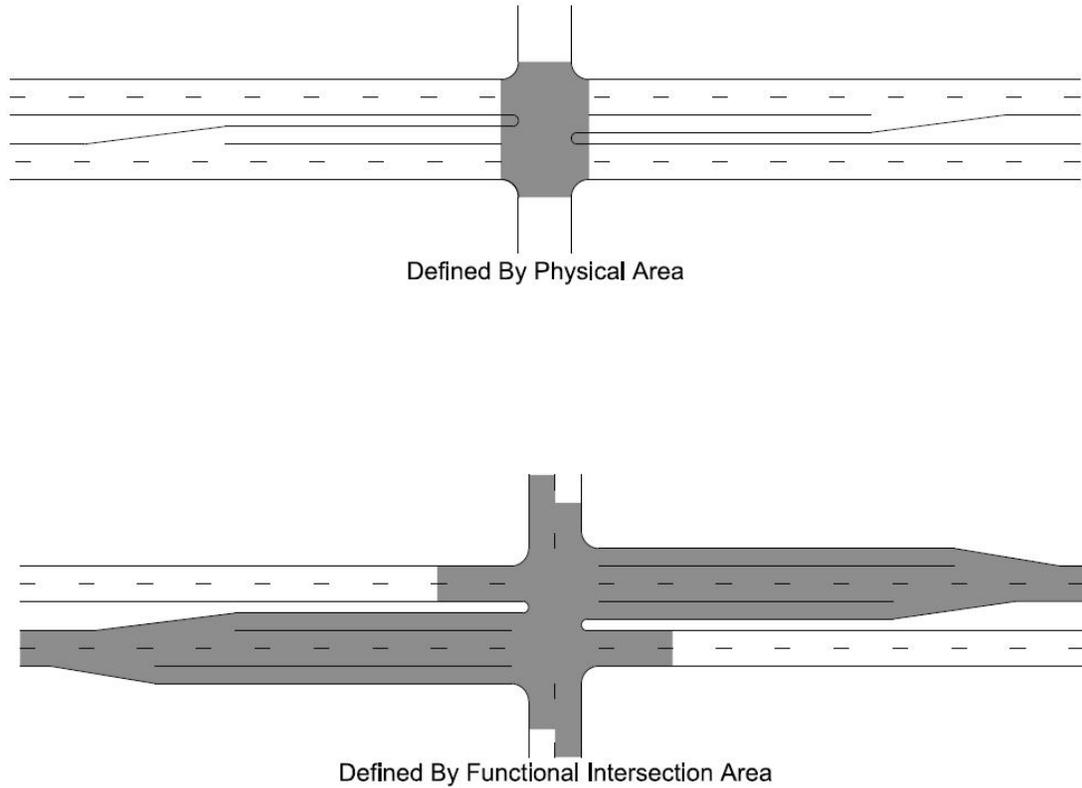
Four basic elements enter into design considerations of intersections:

- 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.

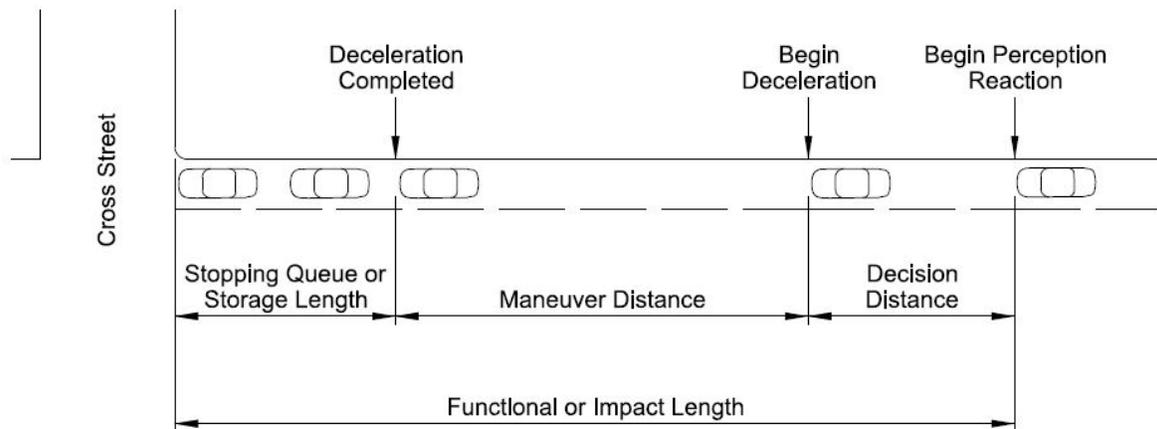
#### 9.1.1 Intersection Functional Area

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 braking 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)*.

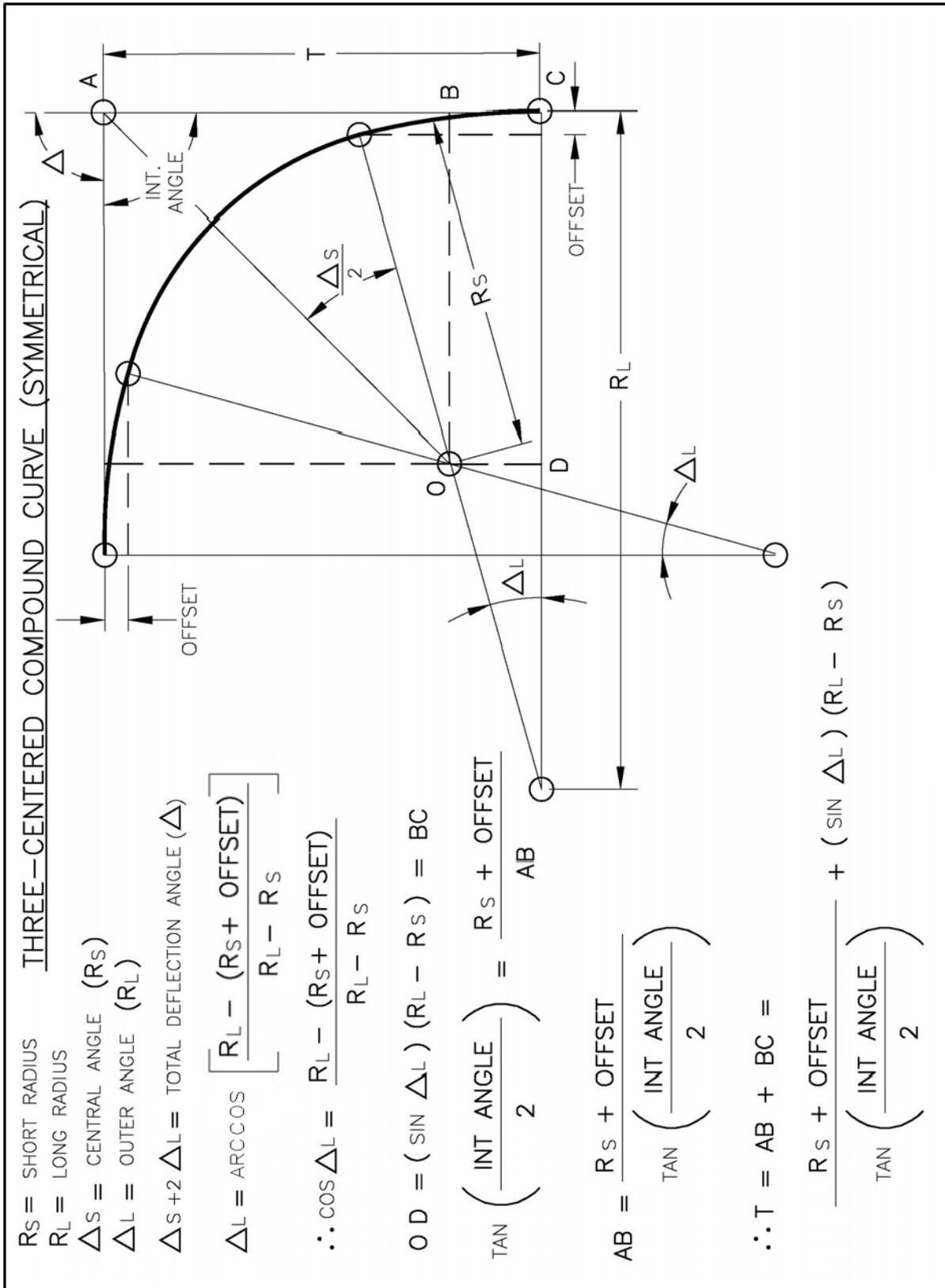


Figure 9-3A Three-Centered Compound Curve (Symmetrical)

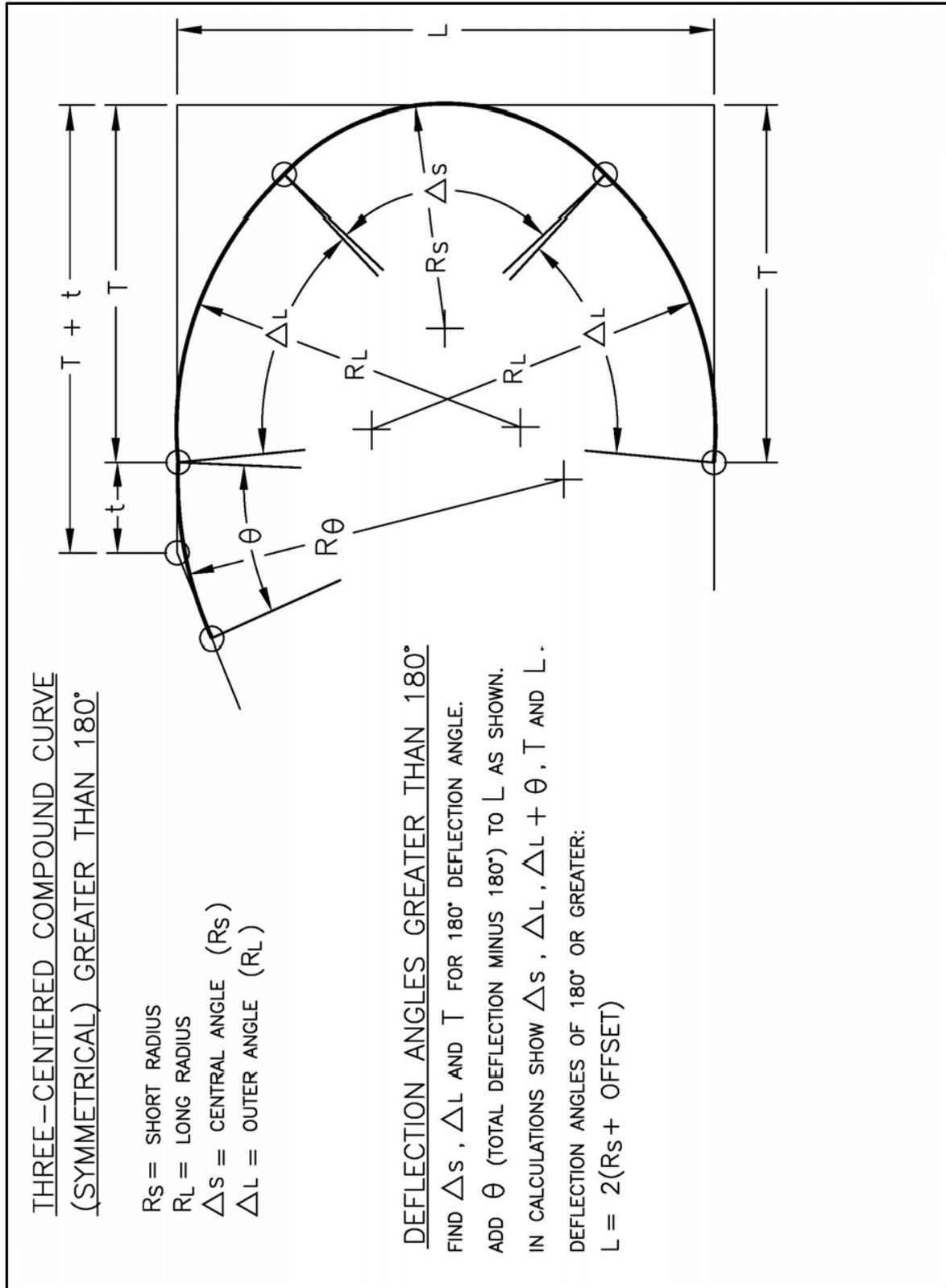


Figure 9-3B Three-Centered Compound Curve (Symmetrical) Greater than 180°

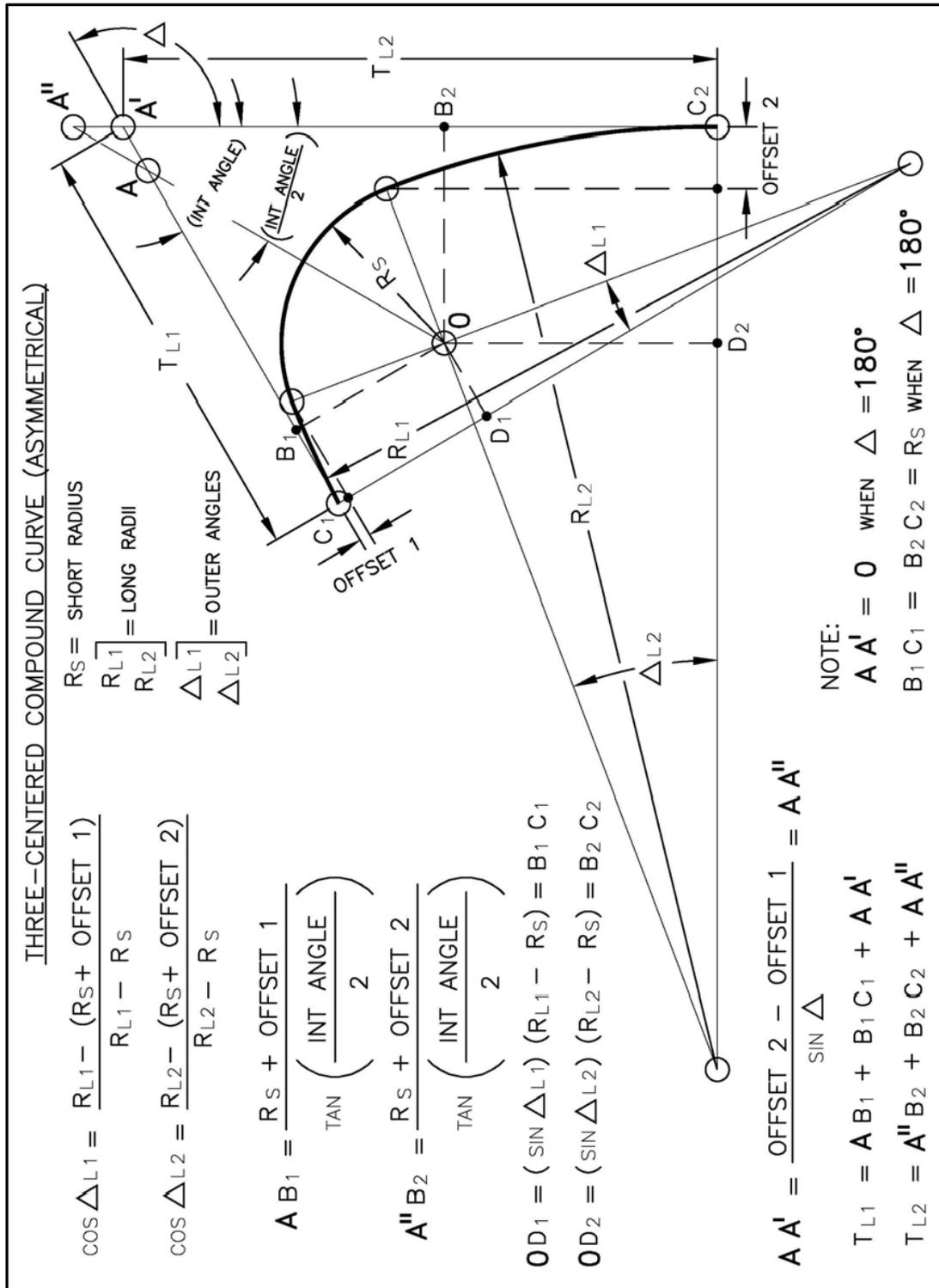


Figure 9-4 Three-Centered Compound Curve (Asymmetrical)

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

**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.

Angle of Turn (degrees)	Design Classification	Three-Centered Curve		Width of Lane (ft)	Approximate Island Size (sq ft)
		Radii (ft)	Offset (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
<p>A - Primarily passenger vehicles; permits occasional design single-unit truck to turn with restricted clearances.</p> <p>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.</p> <p>C - Provides fully for WB-19 [WB-62] design vehicle</p>					
<p>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.</p>					

**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.

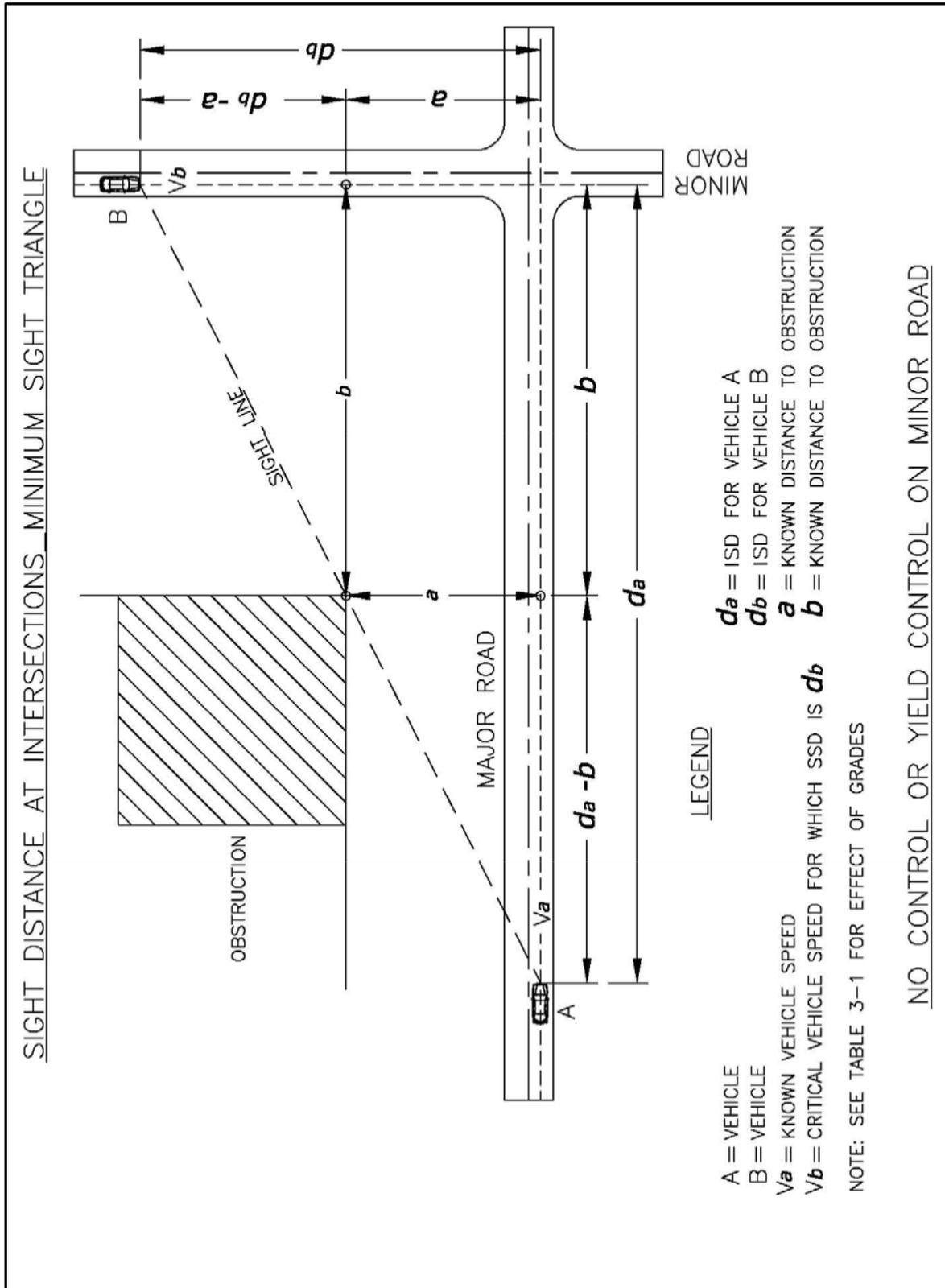


Figure 9-5A Sight Distance at Intersections, Minimum Sight Triangle



### 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

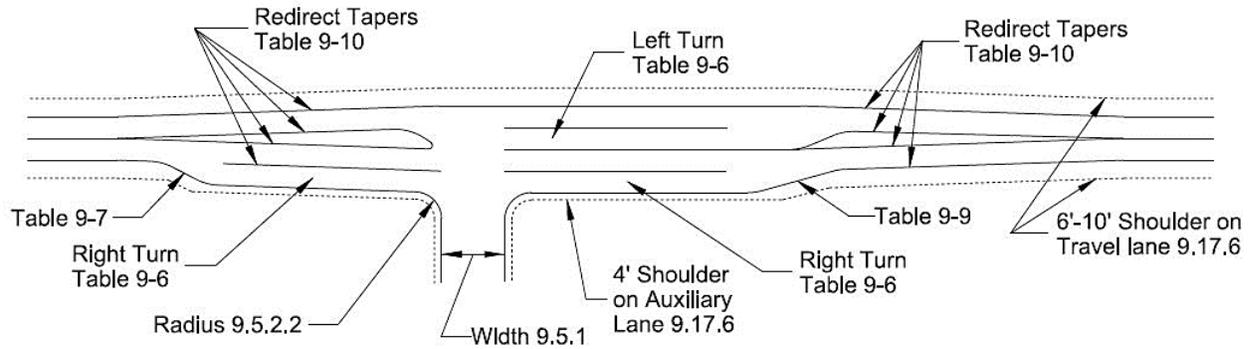
The primary purpose of auxiliary lanes at intersections is to provide storage for turning vehicles, both left and right. A secondary purpose is to provide space for turning vehicles to decelerate from the normal speed of traffic to a stopped position in advance of the intersection or to a safe speed 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.

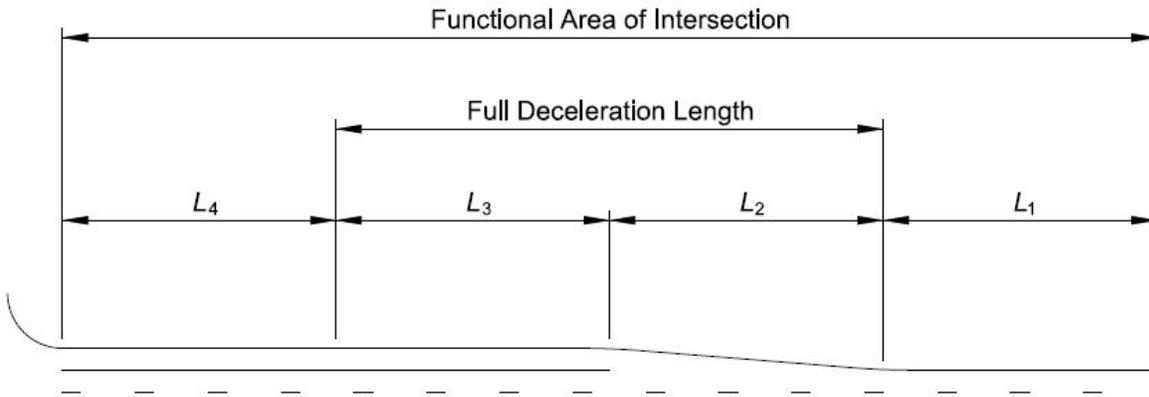
The use and design of speed change lanes differ between functional roadway classifications. Table 9-6 defines the functional roadway classification speed change lane design components by access category. Consult with the Region Traffic Engineer who will determine the need for auxiliary lanes by calculating 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
<b>F-W</b>	Design Must Meet Federal Interstate Standards, and No Less Than E-X		
<b>E-X</b>	Taper + Decel. Length + Storage	Taper + Decel. Length	Accel. Length + Taper
<b>R-A</b>	*Decel. Length + Storage	*Decel. Length	*Accel. Length
<b>R-B</b>	*Decel. Length + Storage	*Decel. Length	*Accel. Length
<b>NR-A</b>	*Decel. Length + Storage	*Decel. Length	*Accel. Length
<b>NR-B</b>	Taper + Storage	Taper + Storage	*Accel. Length
<b>NR-B &gt; 40 MPH</b>	*Decel. Length	*Decel. Length	*Accel. Length
<b>NR-C</b>	Taper + Storage	Taper + Storage	*Accel. Length
<b>NR-C &gt; 40 MPH</b>	*Decel. Length	*Decel. Length	*Accel. Length
<b>F-R</b>	Taper + Storage	Taper + Storage	*Accel. Length
<b>F-R &gt; 40 MPH</b>	*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.



- Notes: L<sub>1</sub> = Distance traveled during perception-reaction time  
 L<sub>2</sub> = Taper distance to begin deceleration and complete lateral movement  
 L<sub>3</sub> = Distance traveled to complete deceleration to a stop  
 L<sub>4</sub> = 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 <sup>a</sup> (Feet)	70	160	275	425	605	820
<sup>a</sup> 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.

<b>Turning Vehicles Per Peak Hour</b>	Below 30	30	60	100	200	300
<b>Required Storage Length (ft)</b>	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 <sup>a</sup>	65 <sup>a</sup>	70 <sup>a</sup>
Taper Ratio <sup>b</sup>	7.5:1	8:1	10:1	12:1	13.5:1	15:1	18.5:1	25:1	25:1	25:1

<sup>a</sup>Uniform 50:1 to 70:1 tapers are recommended where lengths of acceleration lanes exceed 1300 feet.  
<sup>b</sup>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**

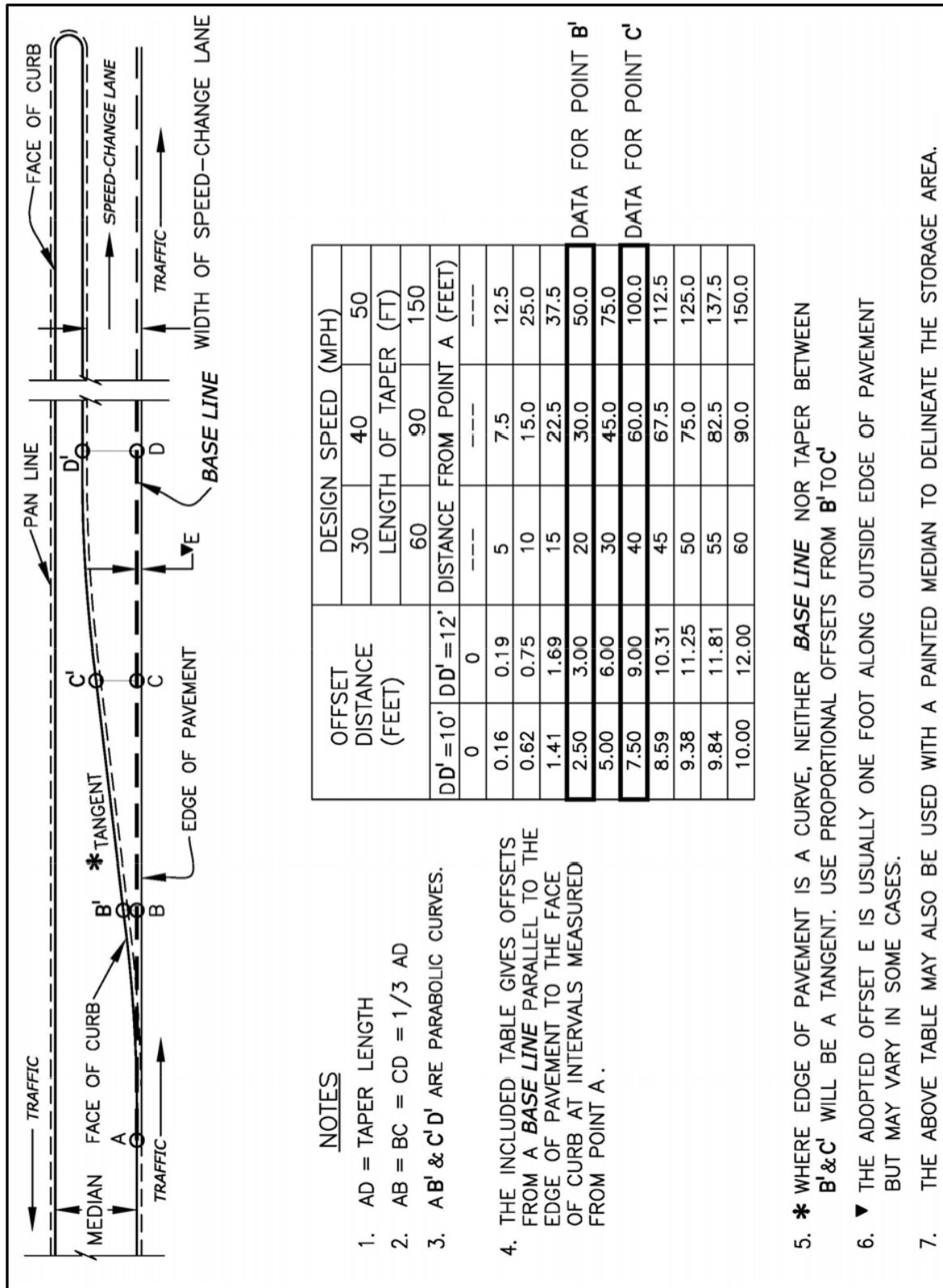


Figure 9-8 Speed-Change Lane Taper for Continuously Curbed Medians

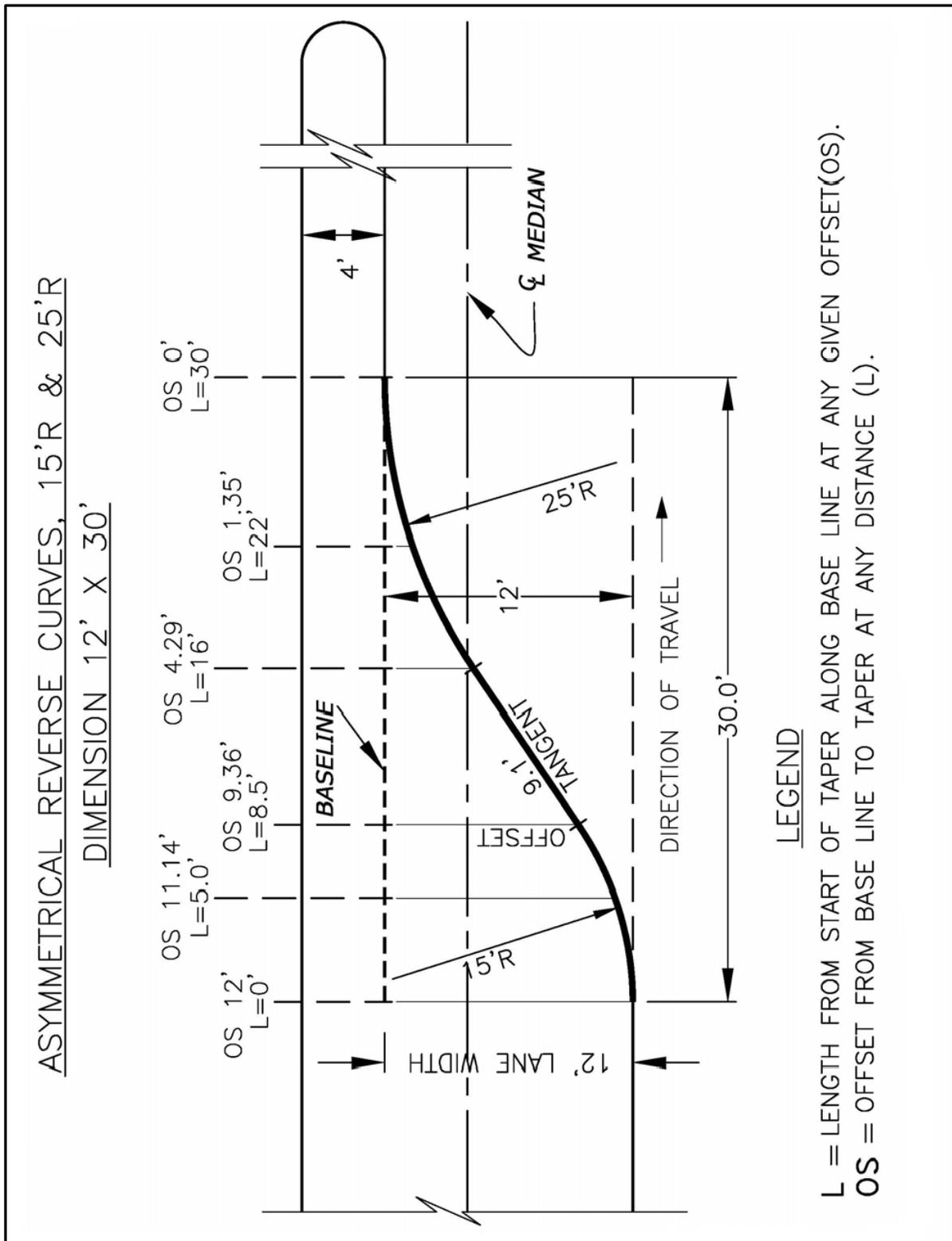


Figure 9-9 (CDOT) Median Bay Taper

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:

$$L = WS \quad [9-1]$$

Where,

$L$  = length of taper, ft  
 $S$  = design speed, mph  
 $W$  = offset, ft

For design speeds less than 45 mph, the *MUTCD* recommends:

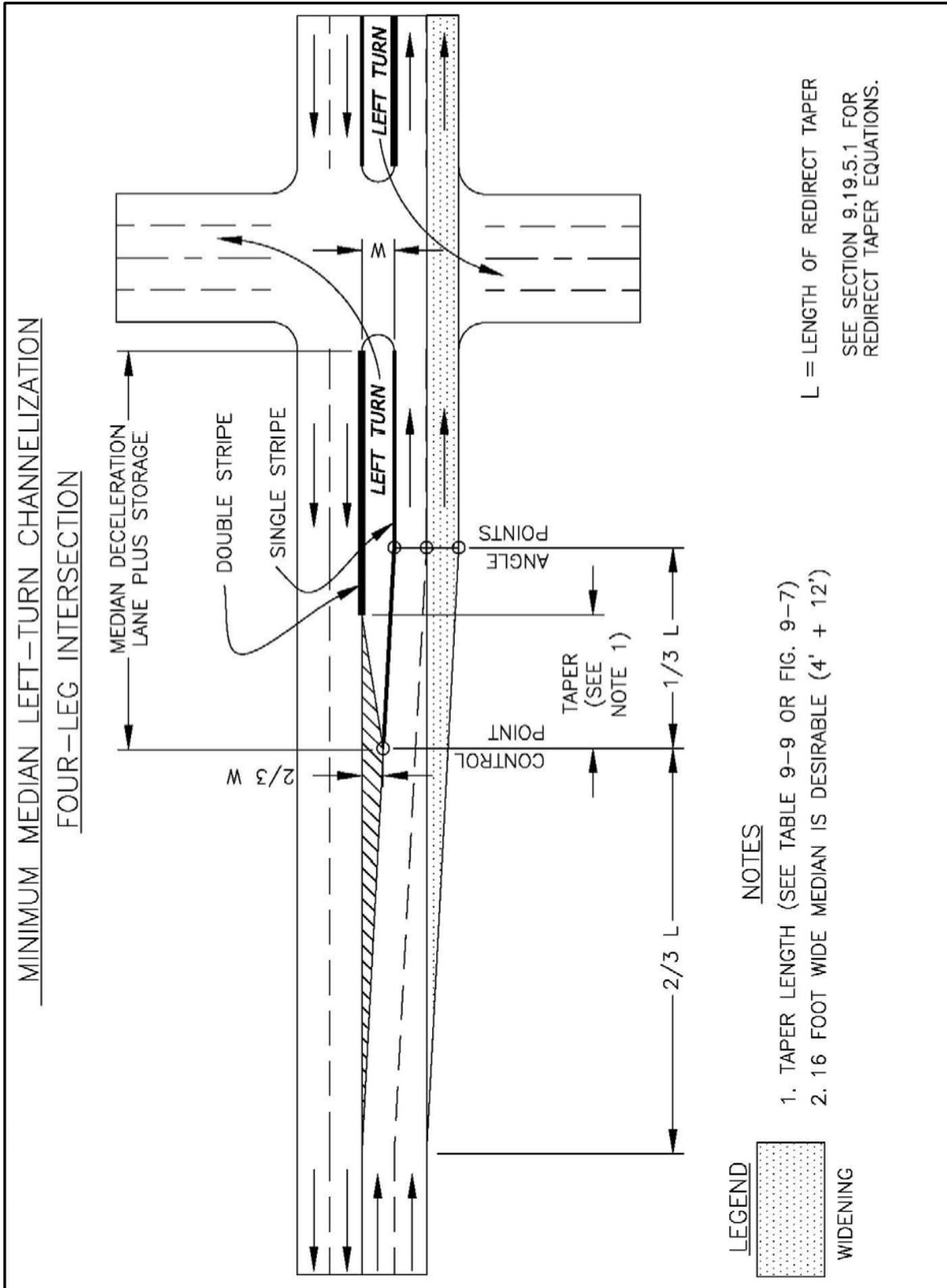
$$L = \frac{WS^2}{60} \quad [9-2]$$

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.

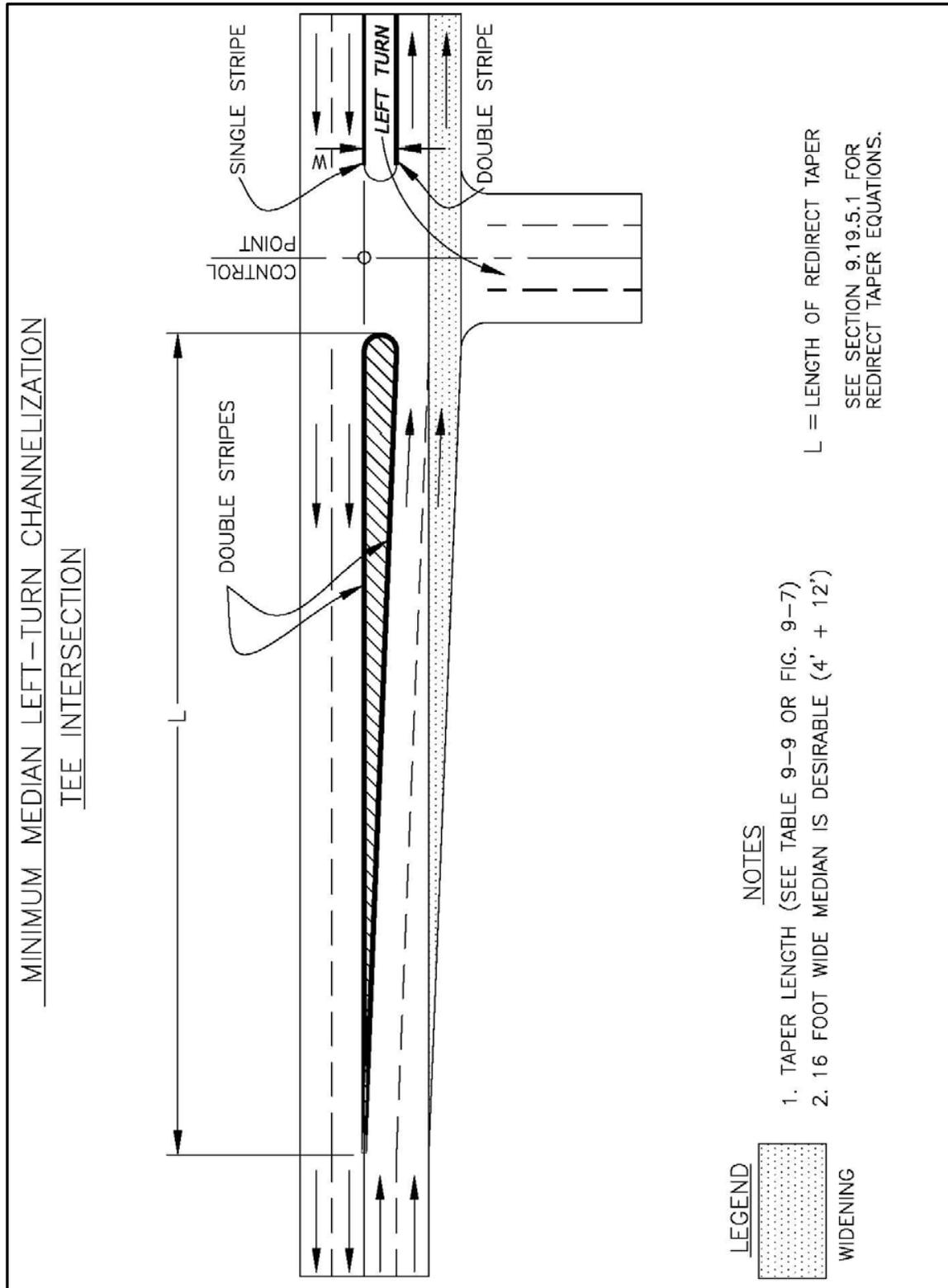
#### **9.17.8 Median Left-Turn Lanes**

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.

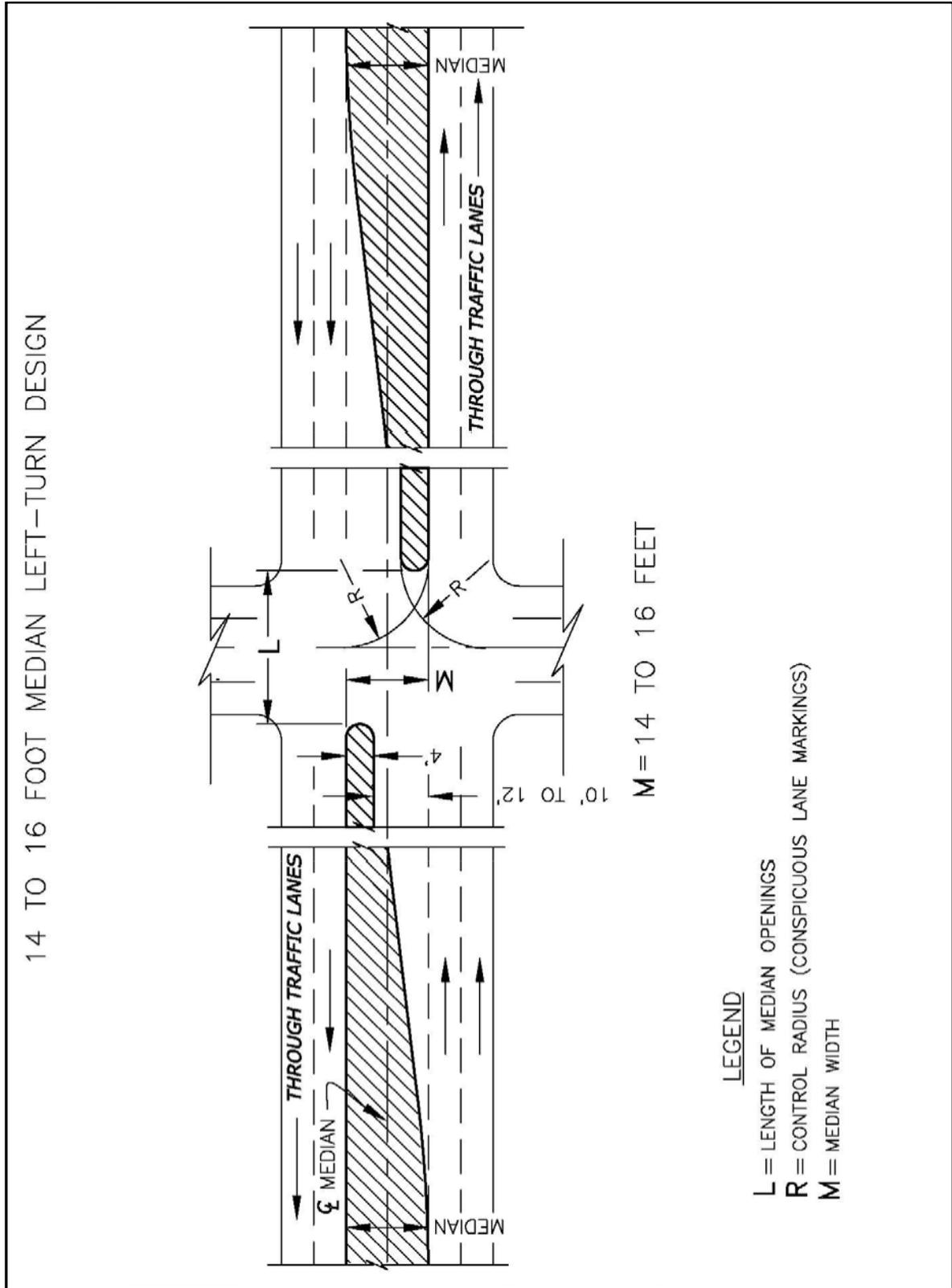


Figure 9-11A [Figure 9-50 of the PGDHS (1)] 14 to 16-Foot Median Left-Turn Design

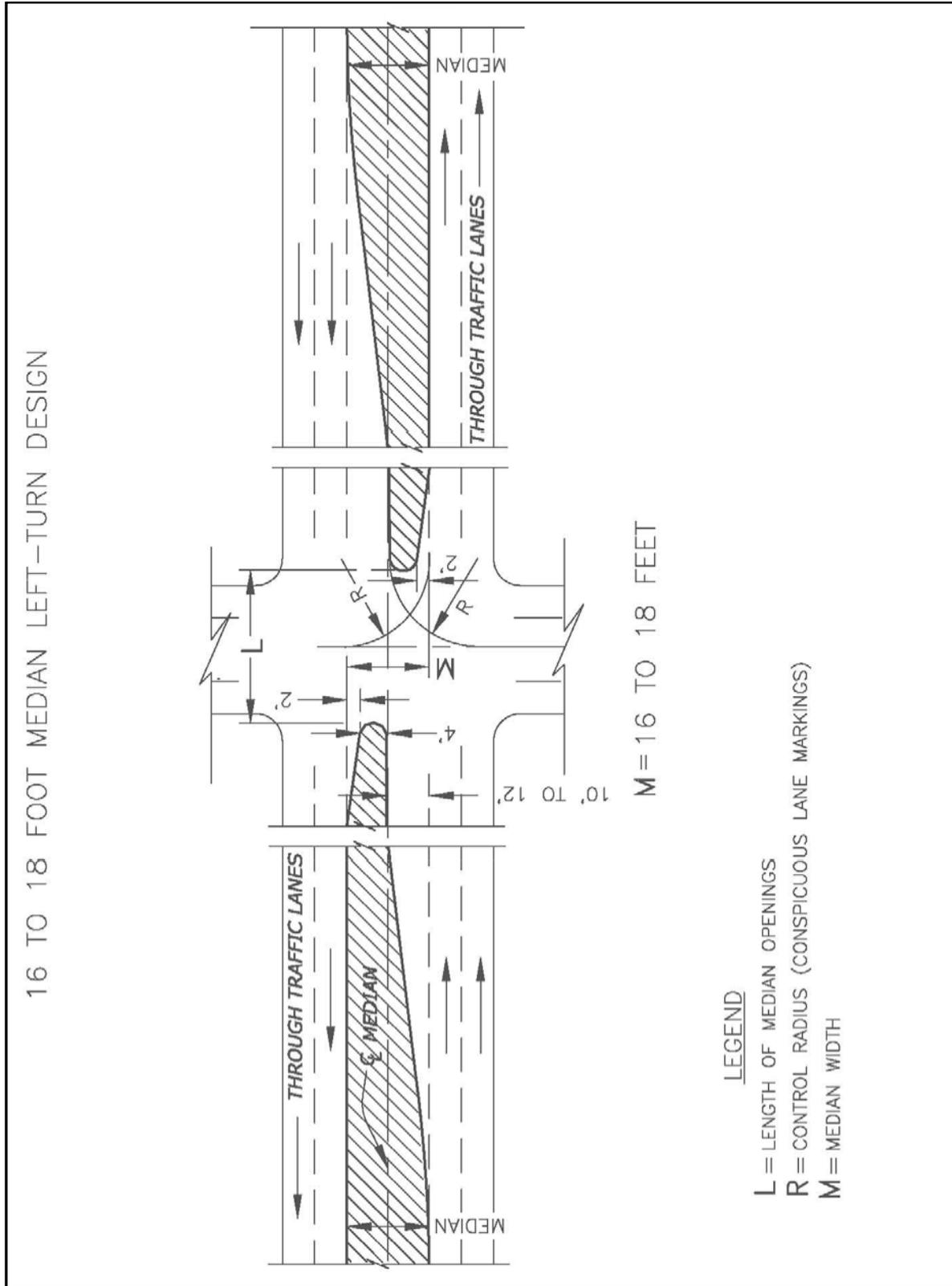


Figure 9-11B [Figure 9-50 of the PGDHS (1)] 16 to 18-Foot Median Left-Turn Design

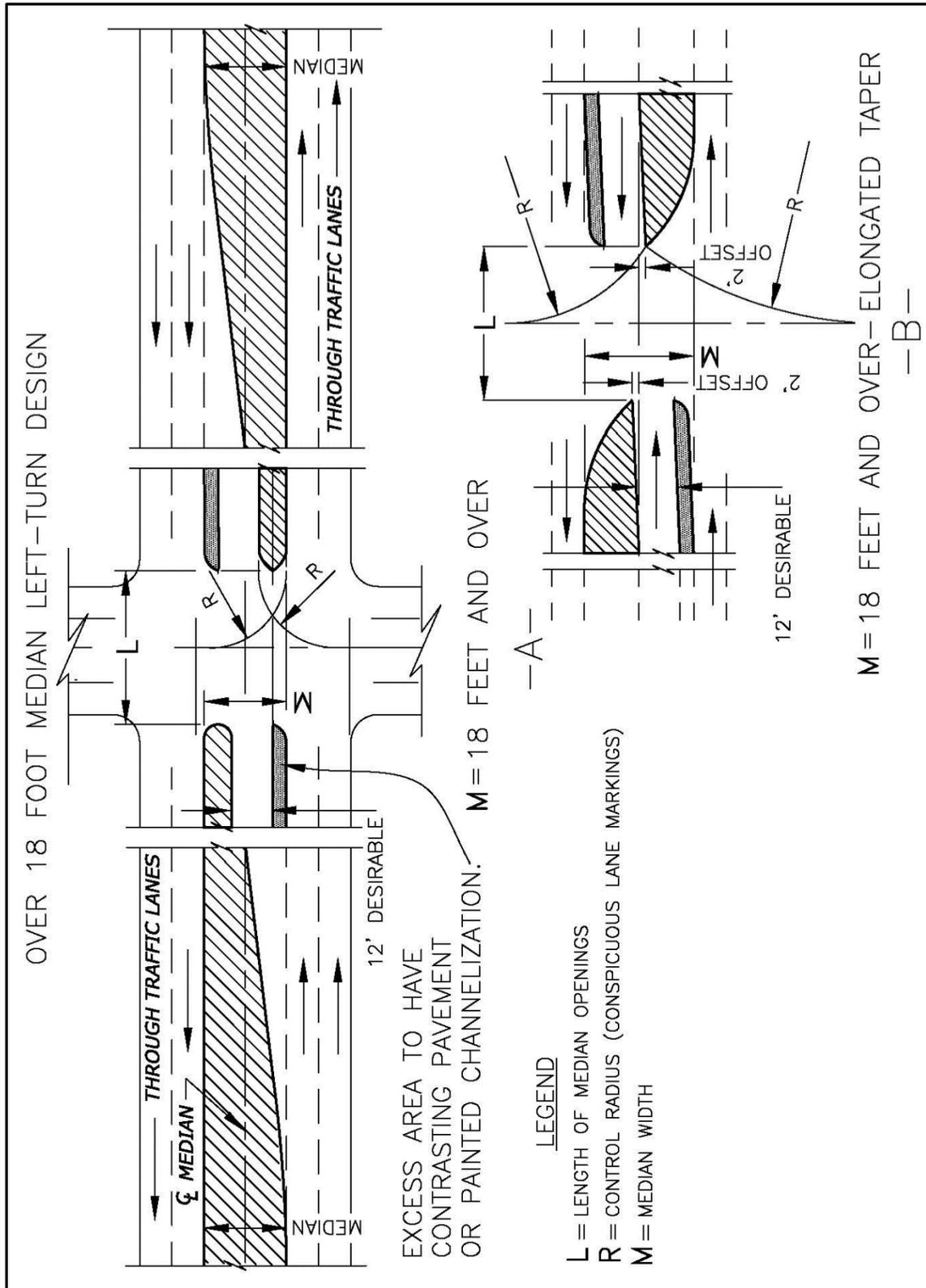


Figure 9-11C [Figure 9-51 of the PGDHS (1)] Median Left Turn Design for Median Width Greater Than 18 Feet

### 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).

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## **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.

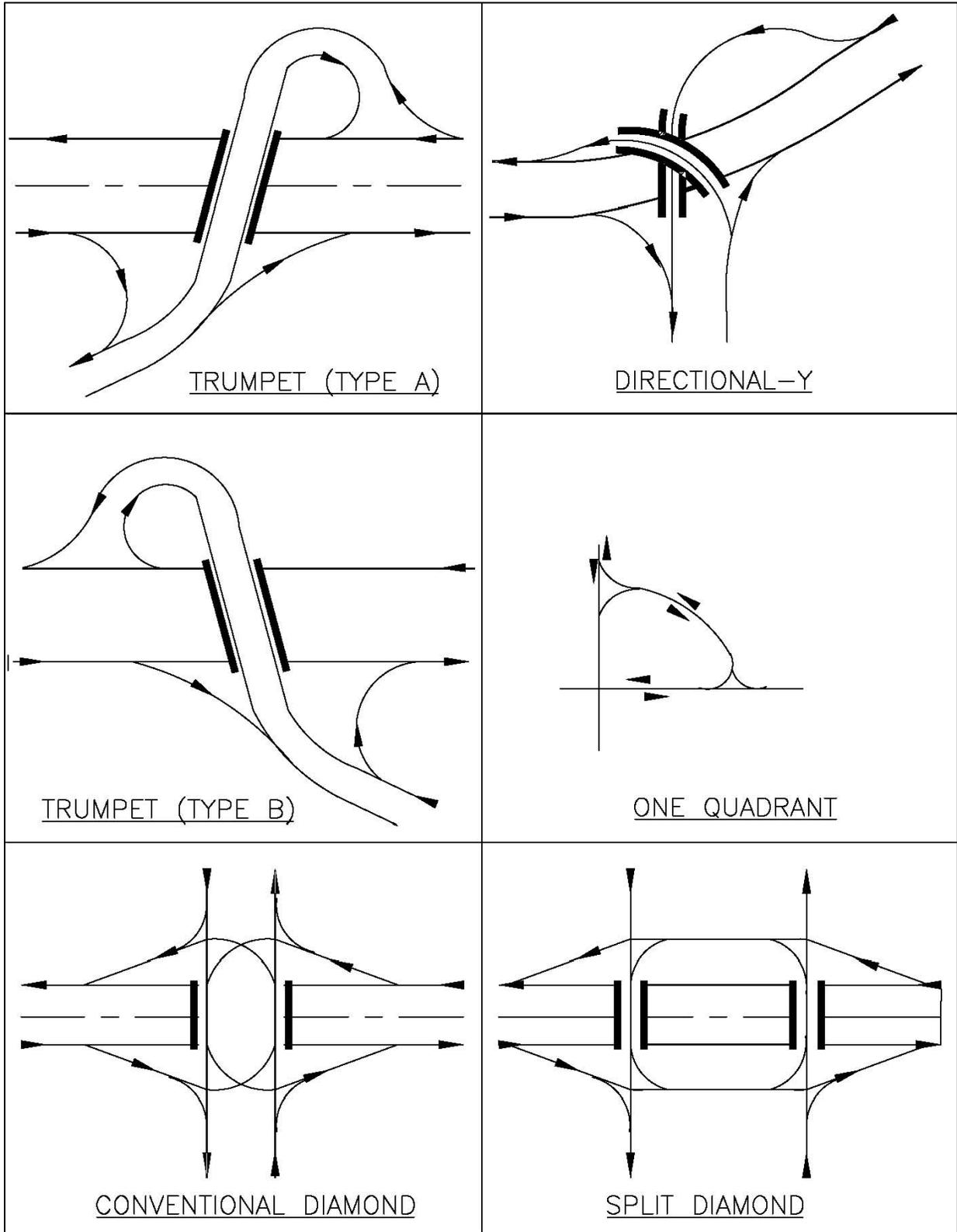
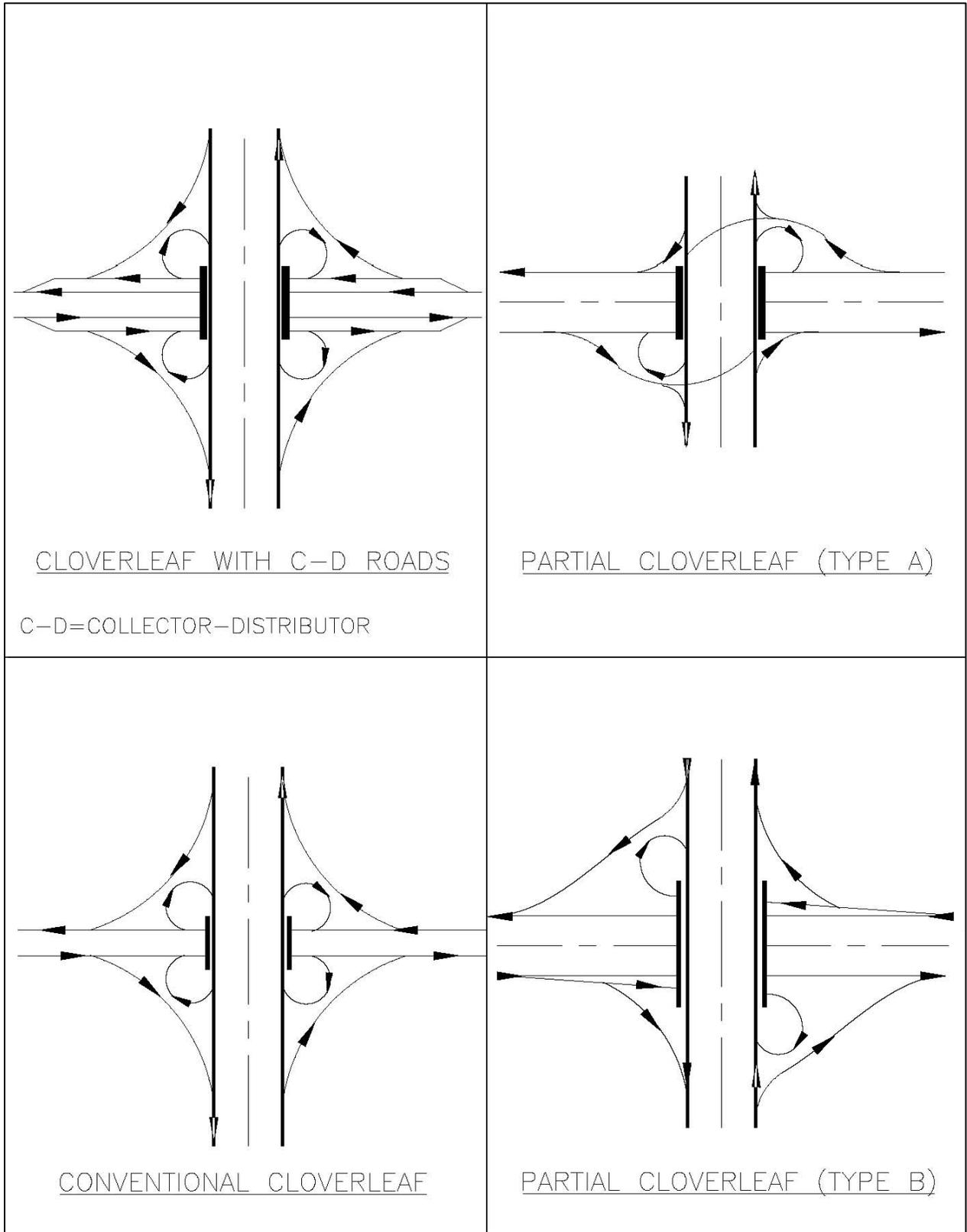
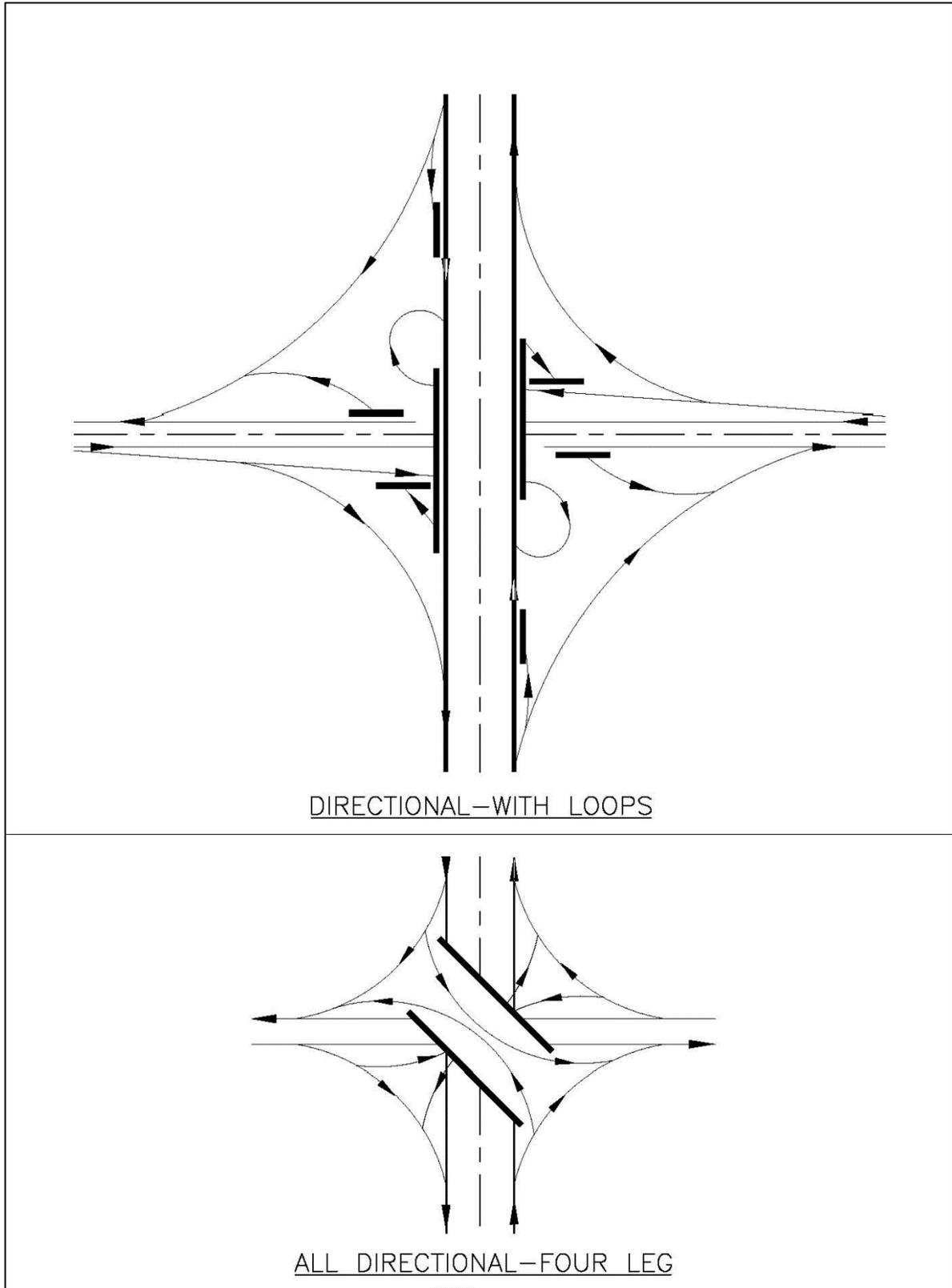


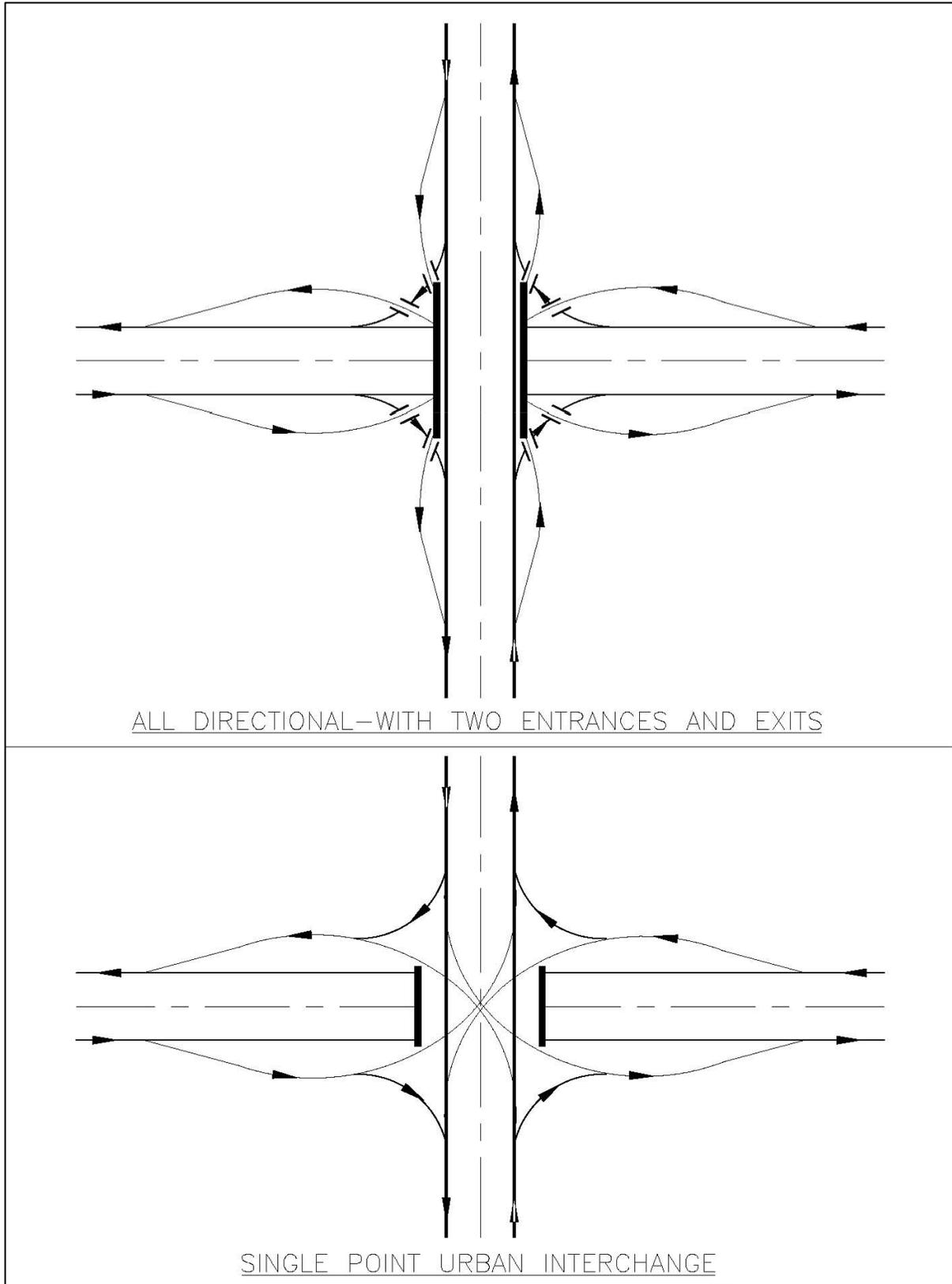
Figure 10-1A Interchange Types



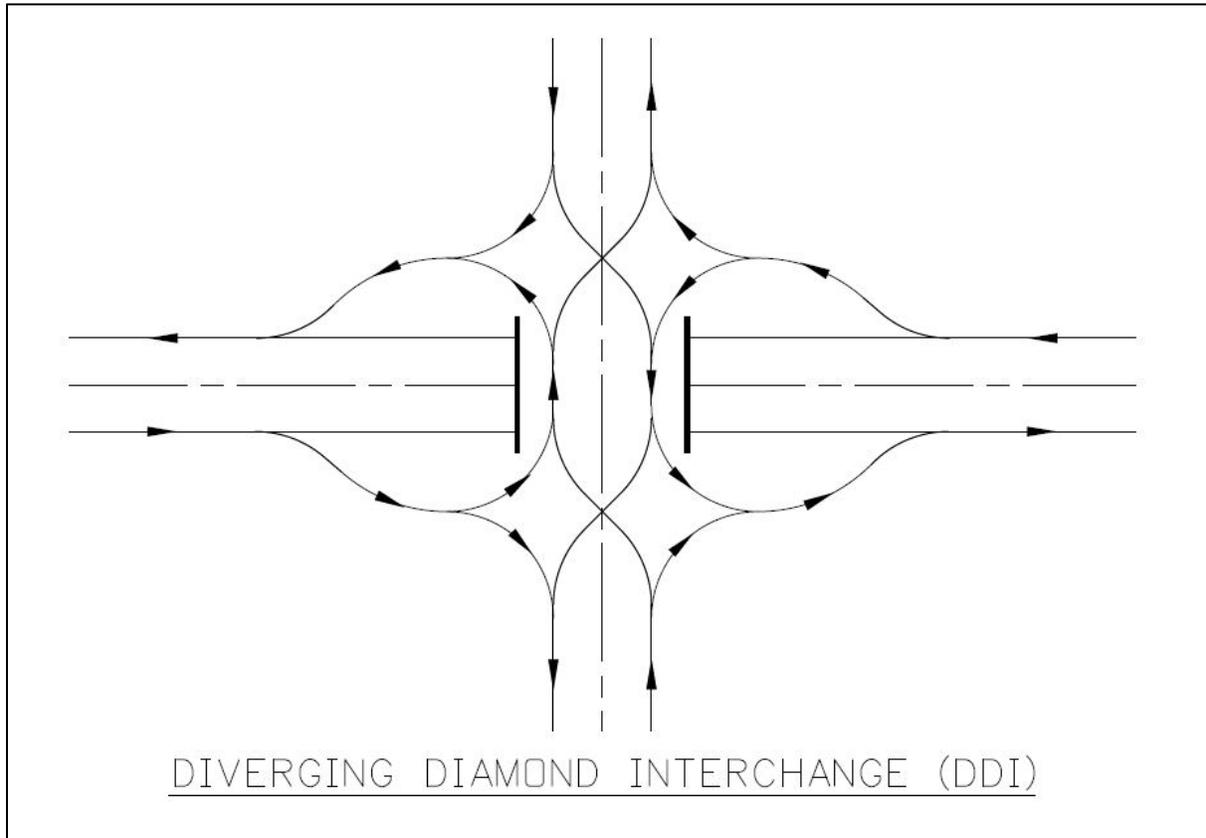
**Figure 10-1B Interchange Types**



**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.

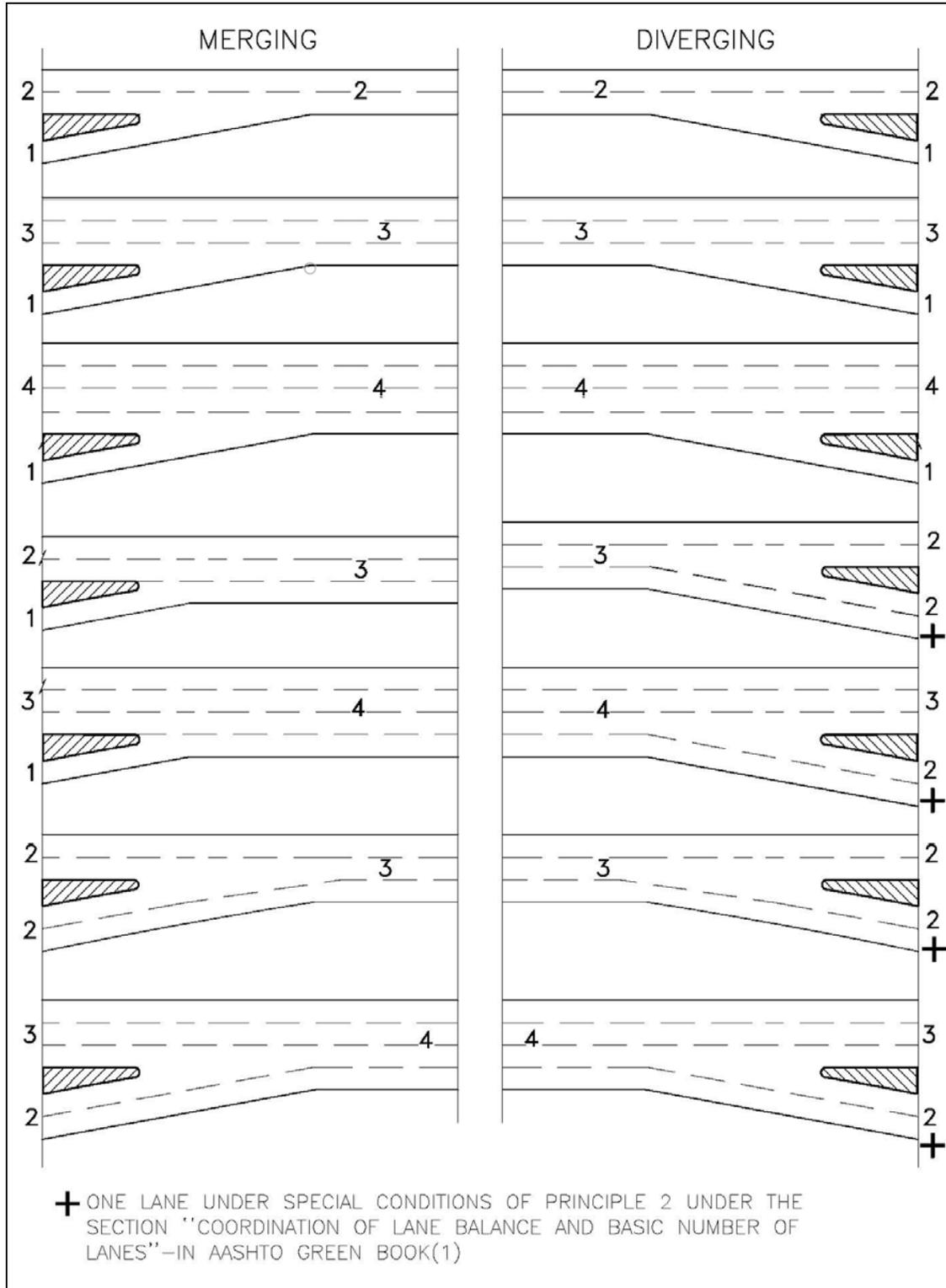


Figure 10-2 [Figure 10-50 of the PGDHS (1)] Typical Examples of Lane Balance

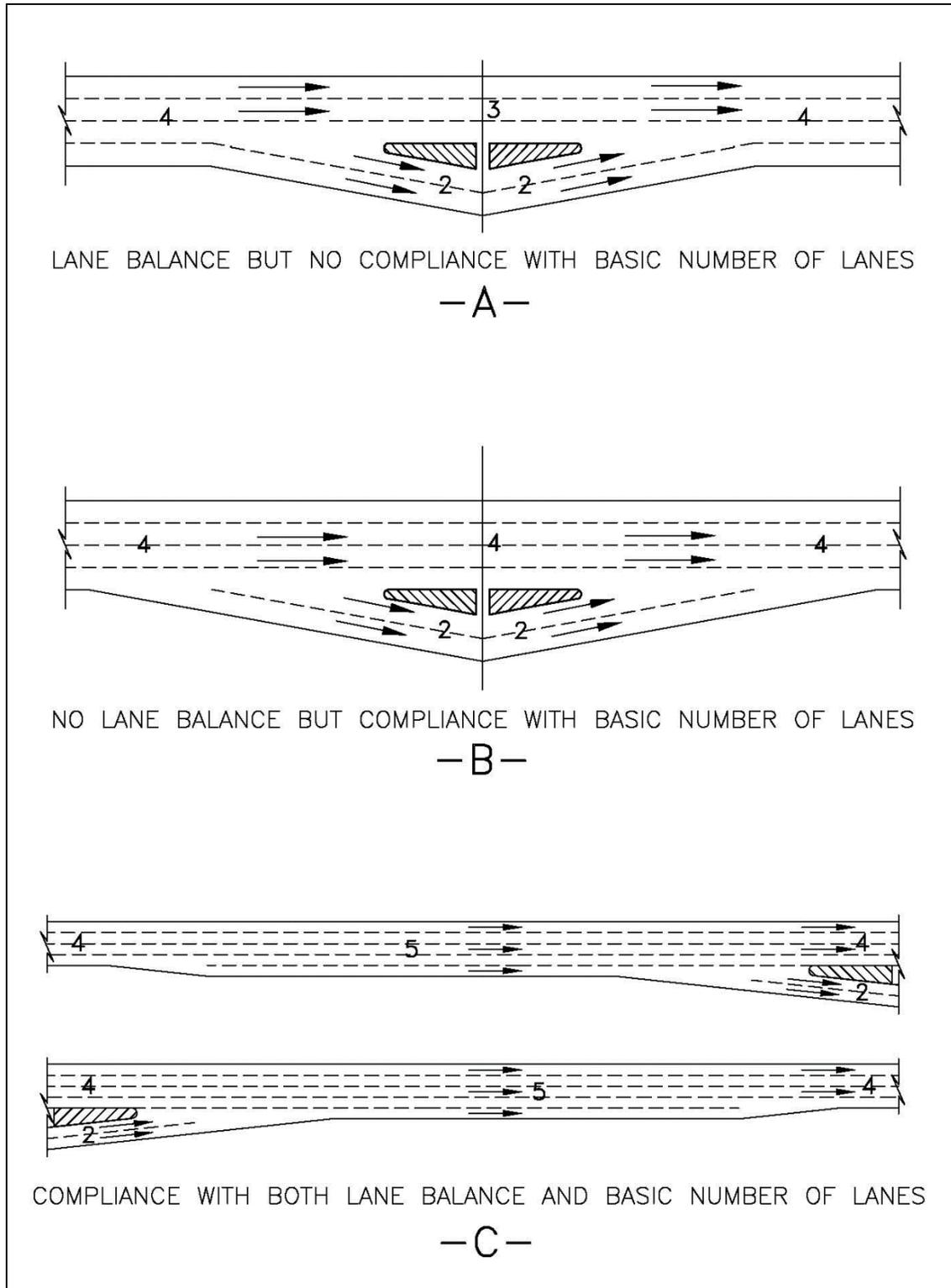


Figure 10-3 [Figure 10-51 of the PGDHS (1)] Coordination of Lane Balance and Basic Number of Lanes

### **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.

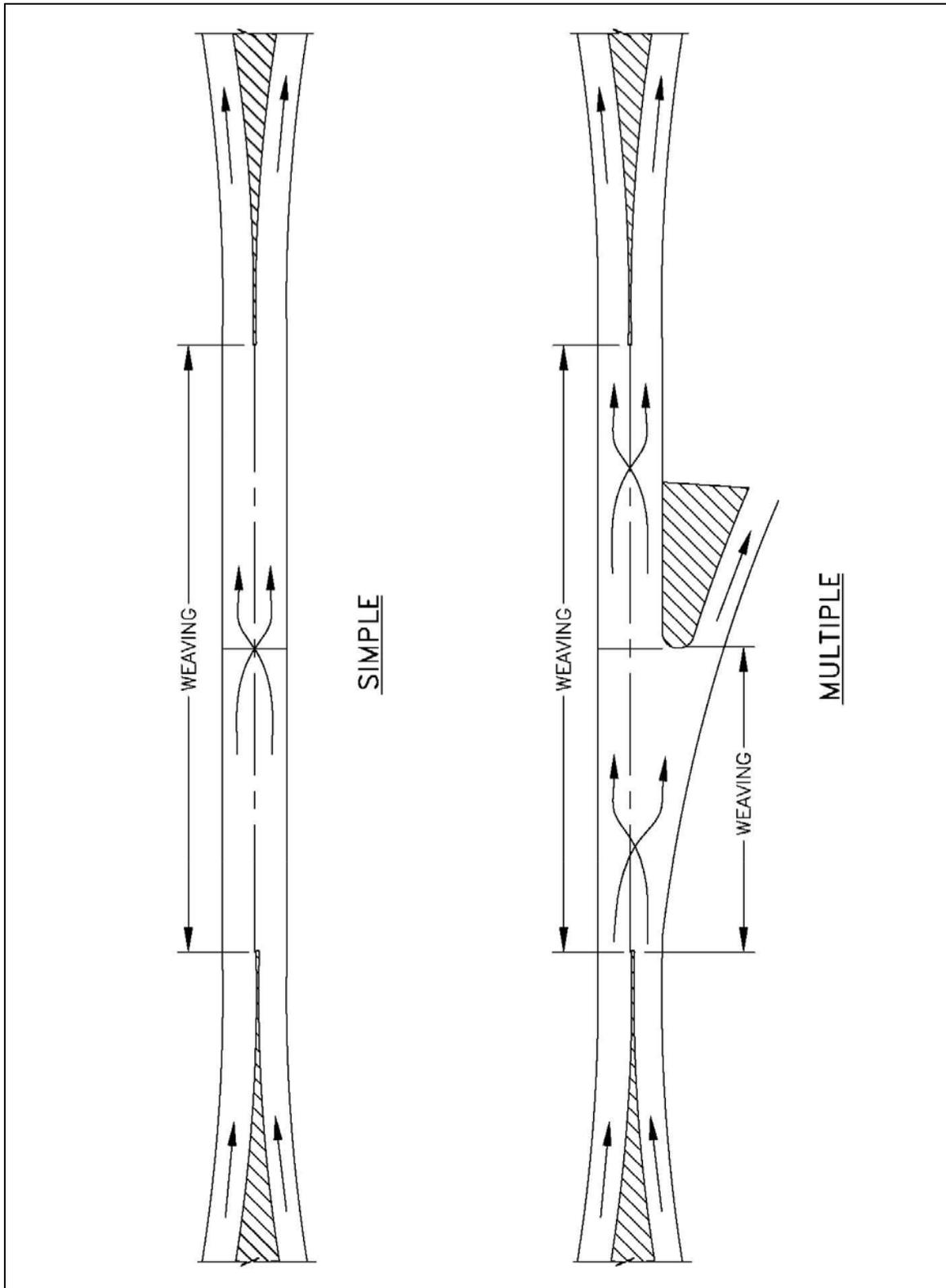


Figure 10-4A Types of Weaving Sections

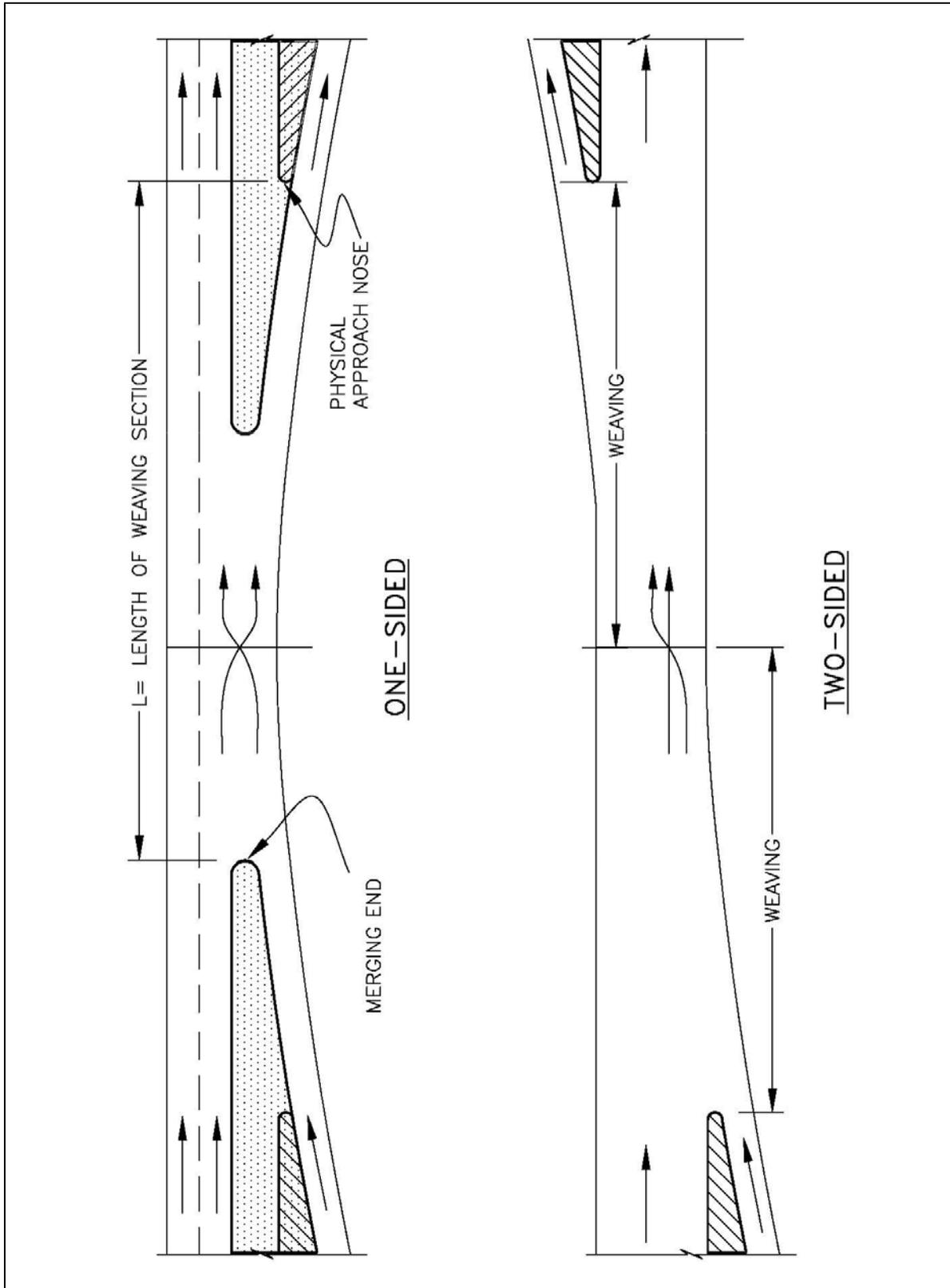


Figure 10-4B Types of Weaving Sections

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.

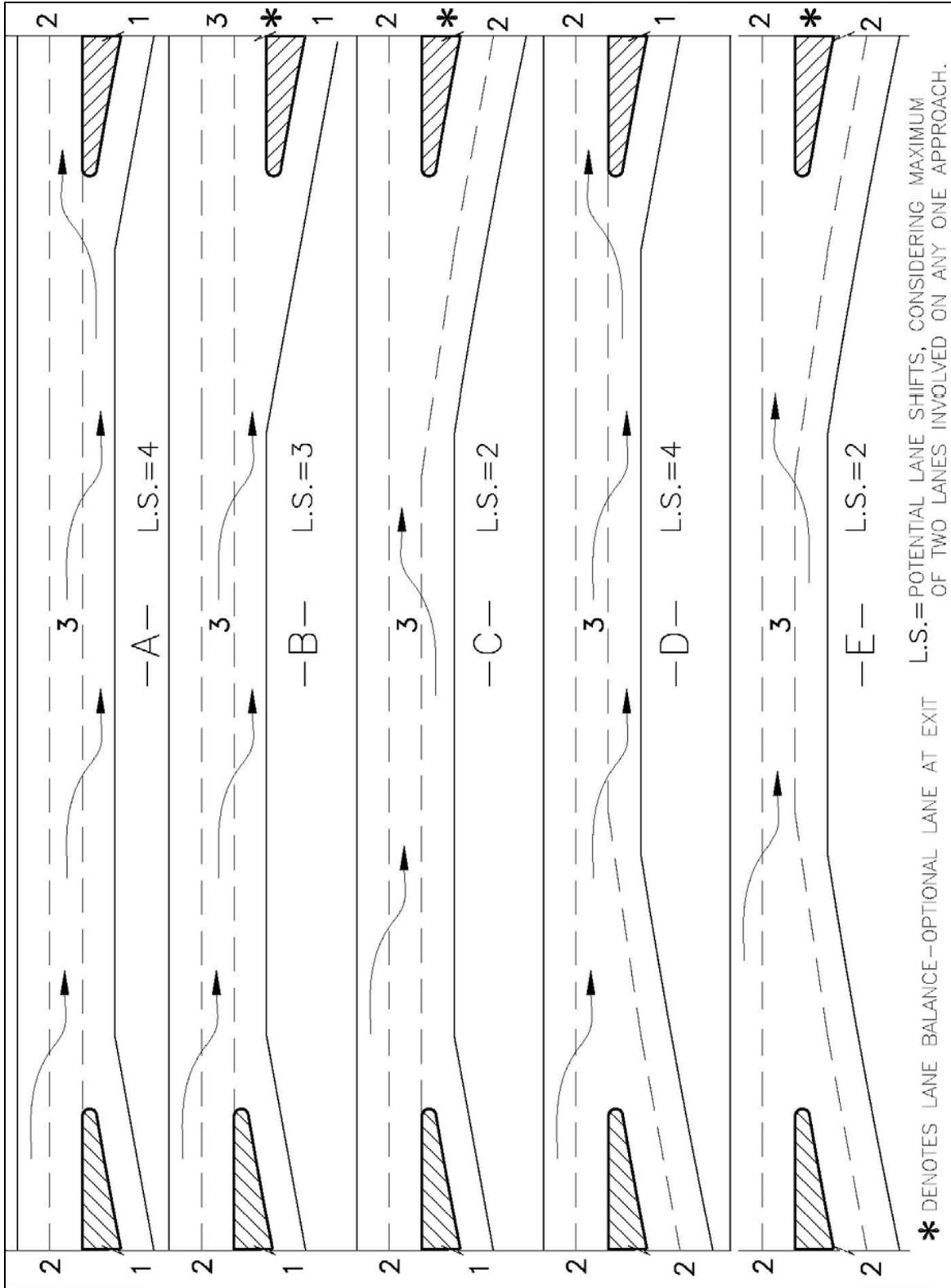


Figure 10-5A (CALTRANS 504. 7B (4)) Lane Configuration of Weaving Sections

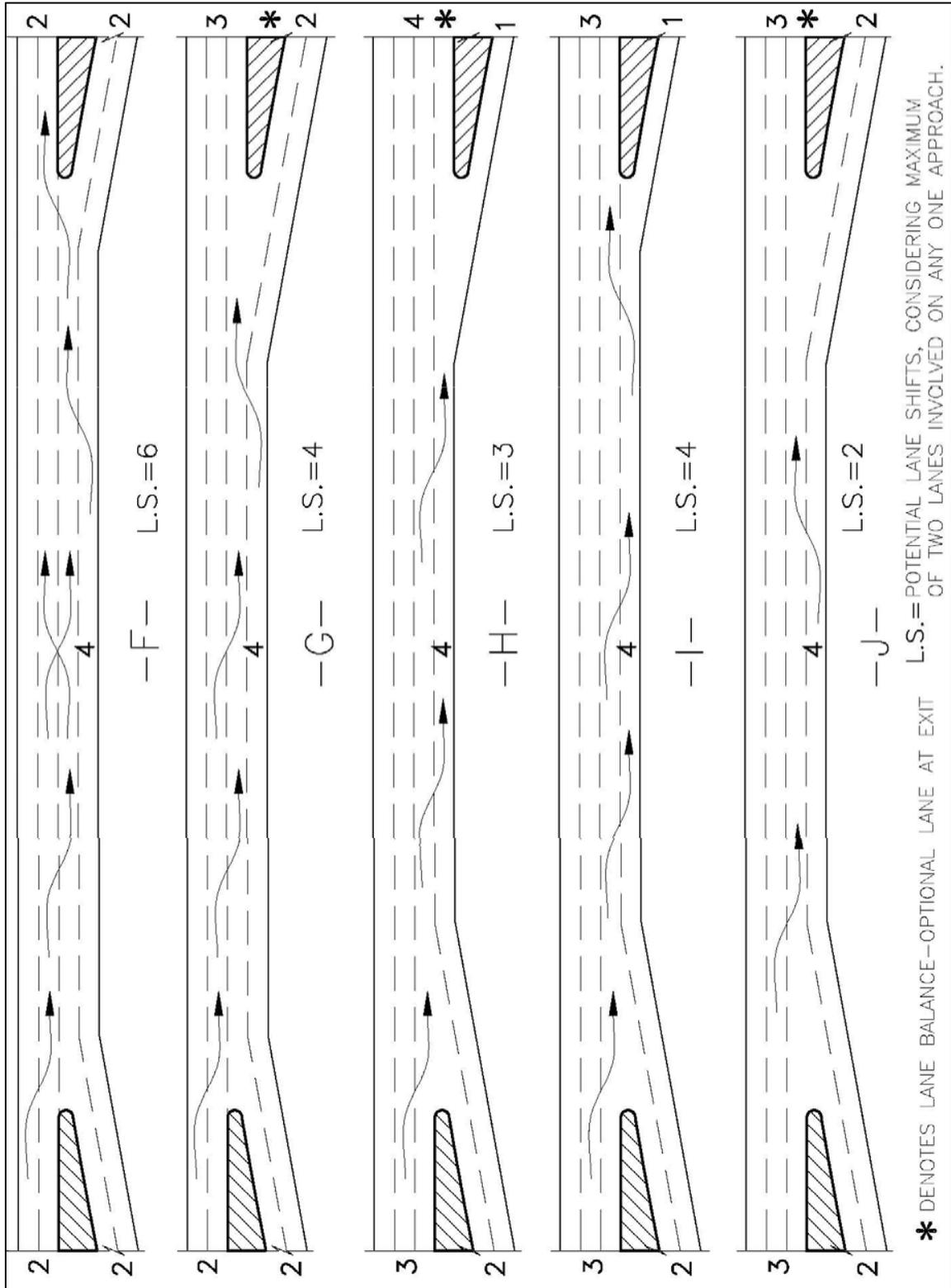


Figure 10-5B (CALTRANS 504. 7B (4)) Lane Configuration of Weaving Sections

### **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 <b>Table 3-7 in the PGDHS (1)</b>									
* 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.

<b>Design Speed of Exit or Entrance Curve (mph)</b>	<b>Maximum Algebraic Difference in Cross Slope at Crossover Line (%)</b>
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), in particular, 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.

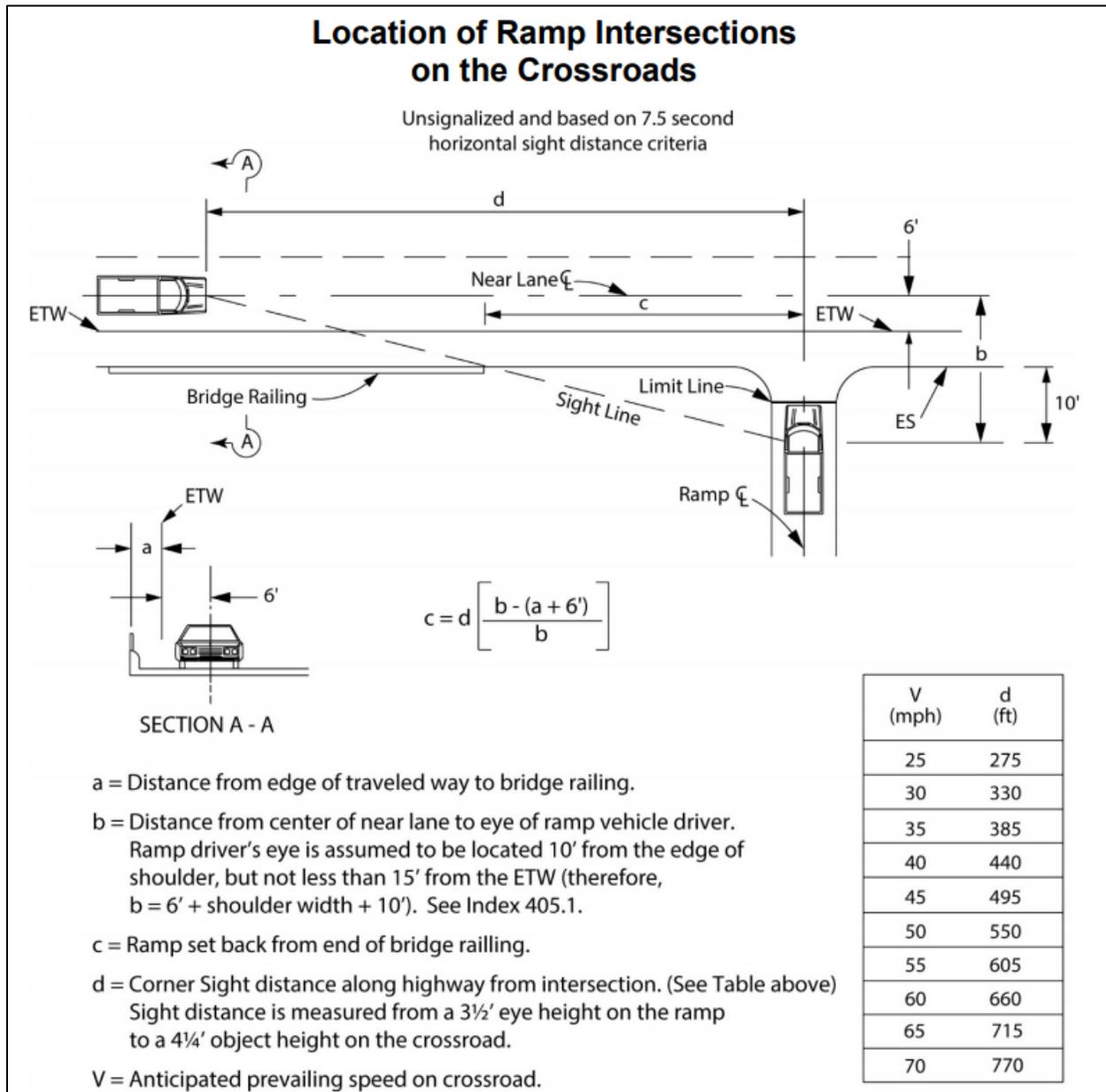


Figure 10-6 (CALTRANS 504.3I (4)) Location of Ramp Intersections on the Cross Road

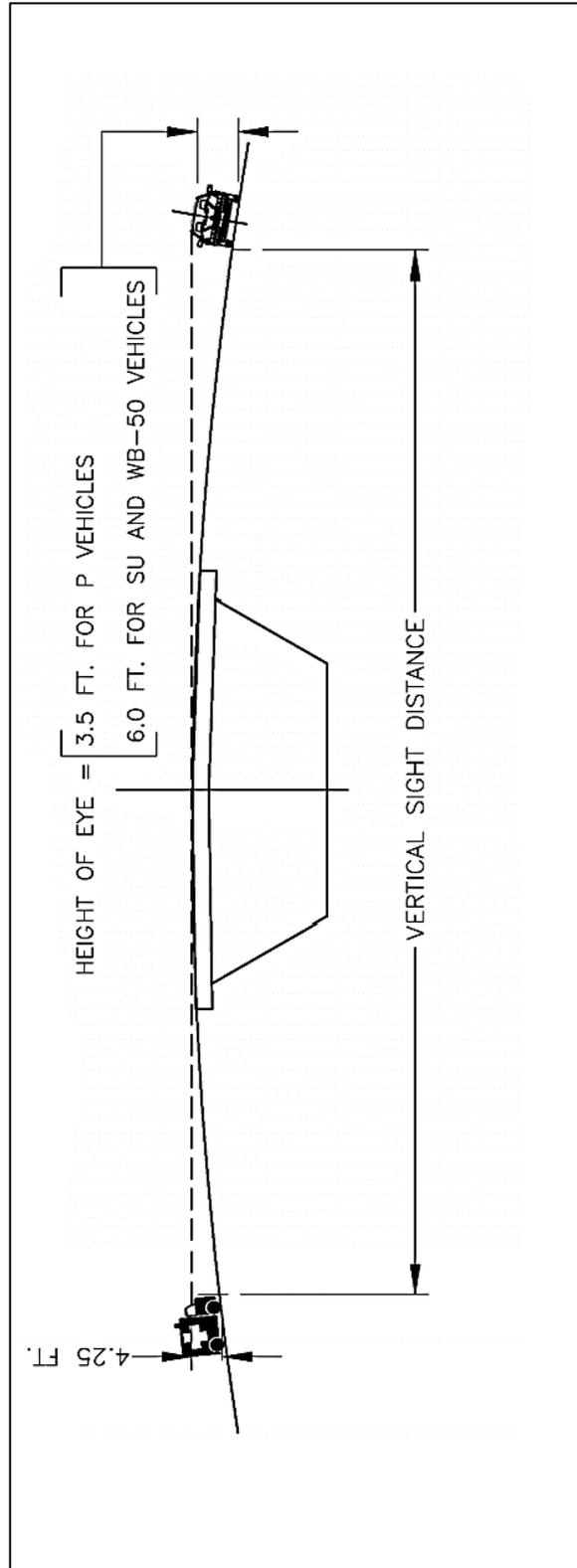
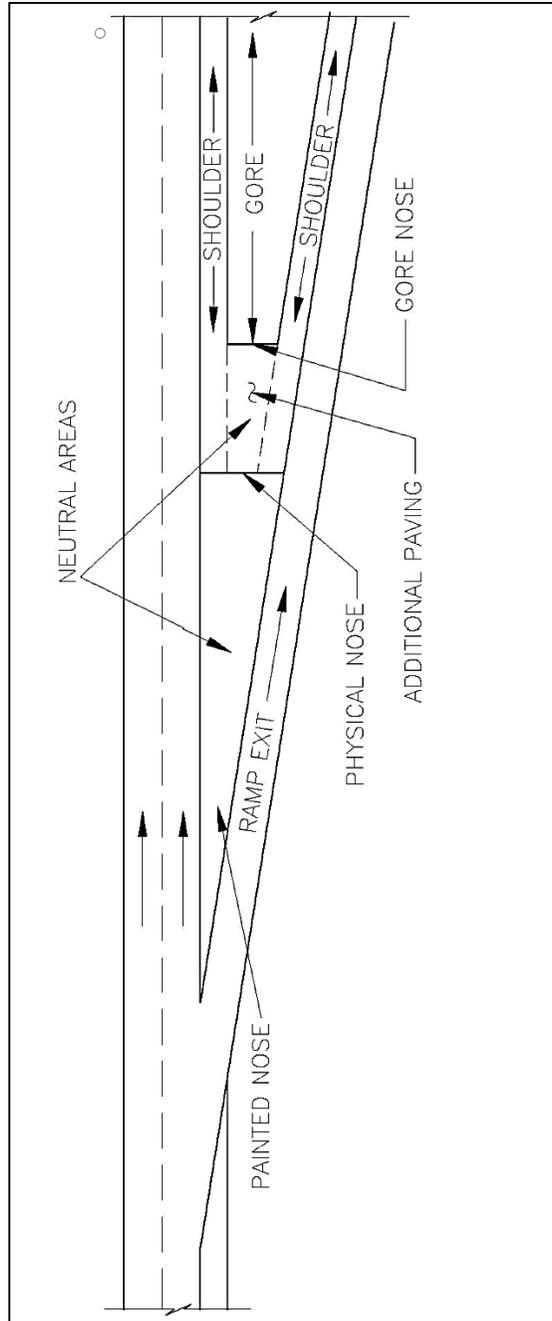


Figure 10-7 Measurement of Sight Distance at Ramp Terminals with Stop Controls

**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.

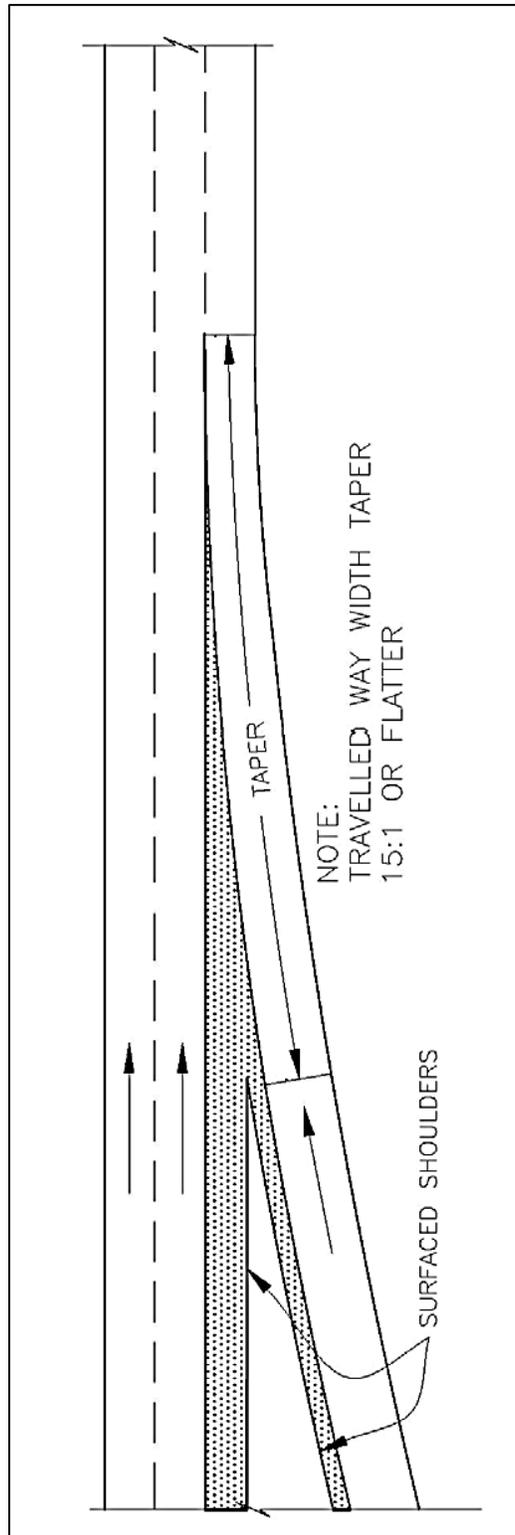


Figure 10-9 [Figure 10-63 of the PGDHS (1)] Traveled Way Narrowing on Entrance Ramps

## 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.

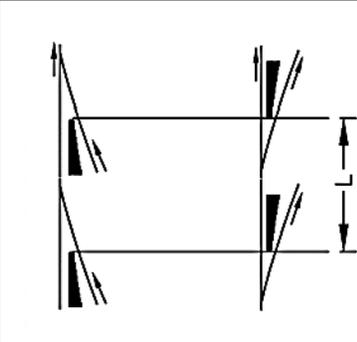
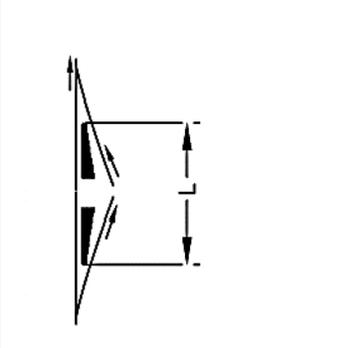
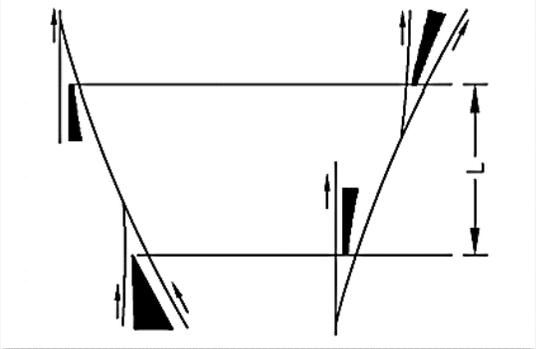
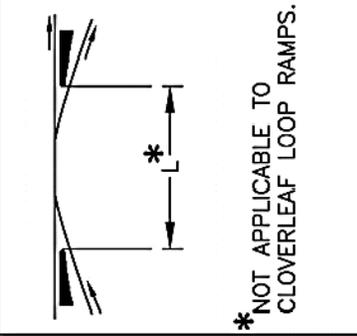
Extreme care should be exercised to avoid left-hand entrances and exits in the design of interchanges. Even in the case of major forks and branch connections, the less significant roadway should exit and enter on the right. See the discussion on route continuity in Chapter 10 of the *PGDHS (1)*.

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.

EN-EN OR EX-EX	EX-EN	TURNING ROADWAYS	EN-EX (WEAVING)
			 <p data-bbox="638 195 703 531">* NOT APPLICABLE TO CLOVERLEAF LOOP RAMP.</p>
FULL FREEWAY	FULL FREEWAY	SYSTEM INTERCHANGE	SYSTEM TO SERVICE INTERCHANGE
		CDR OR FDR	FULL FREeway OR CDR OR FDR
MINIMUM LENGTHS MEASURED BETWEEN SUCCESSIVE RAMP TERMINALS (FEET)			
1000	500	800	2000
		600	1600
			1000

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 <sub>A</sub> )	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' <sub>A</sub> )								
		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

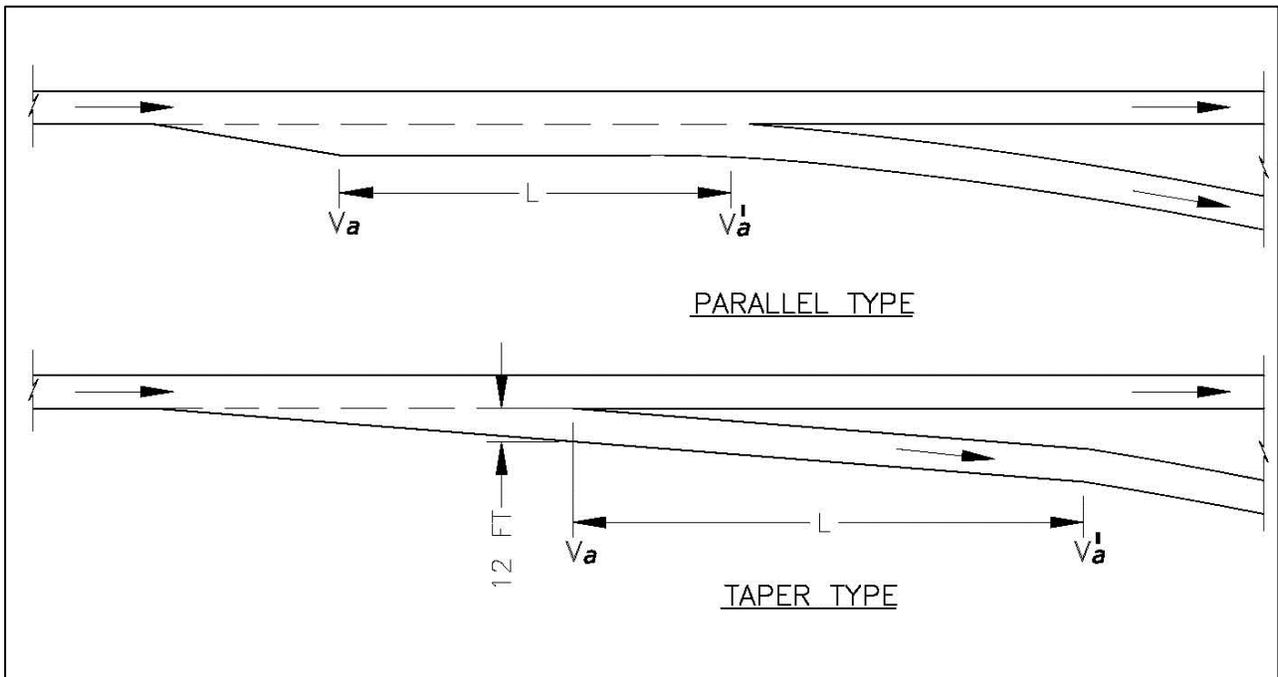


Table 10-3 [Table 10-5 of the PGDHS (1)] Minimum Deceleration Lengths for Exit Terminals With Flat Grades of 2 Percent or Less

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.

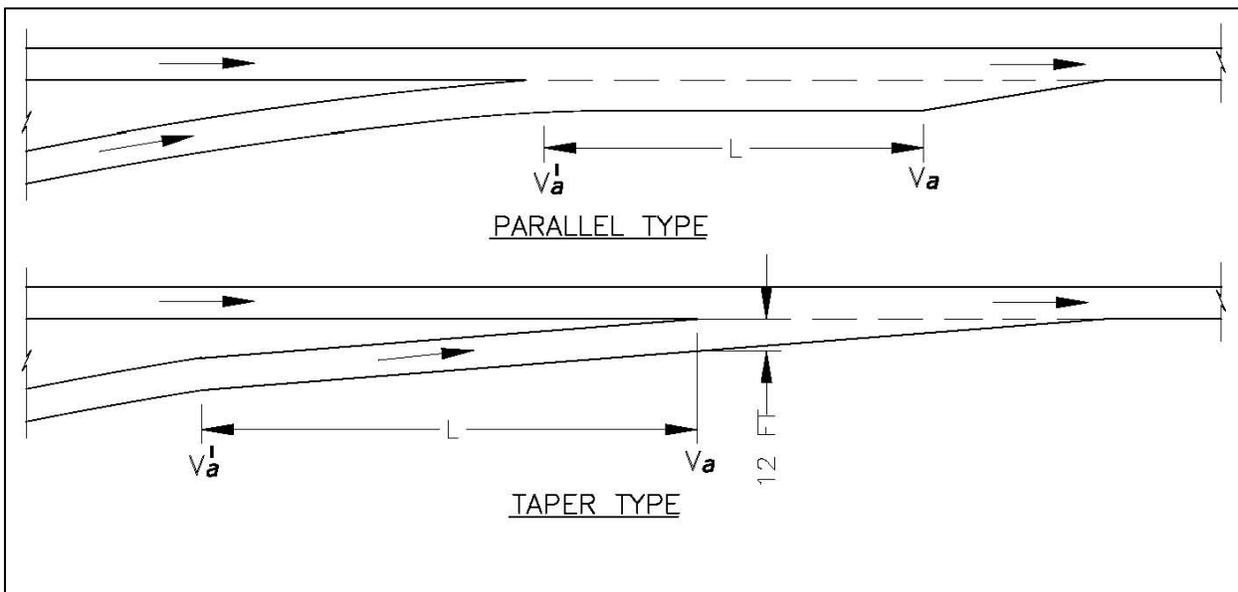


Table 10-4 [Table 10-3 of the PGDHS (1)] Minimum Acceleration Lengths for Entrance Terminals With Flat Grades of 2 Percent or Less

DESIGN SPEED OF HIGHWAY (mph)	DECELERATION LANES				
	Ratio of Length on Grade to Length on Level for Design Speed of Turning Curve (mph) <sup>a</sup>				
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) <sup>*</sup>				
	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

<sup>\*</sup> 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**

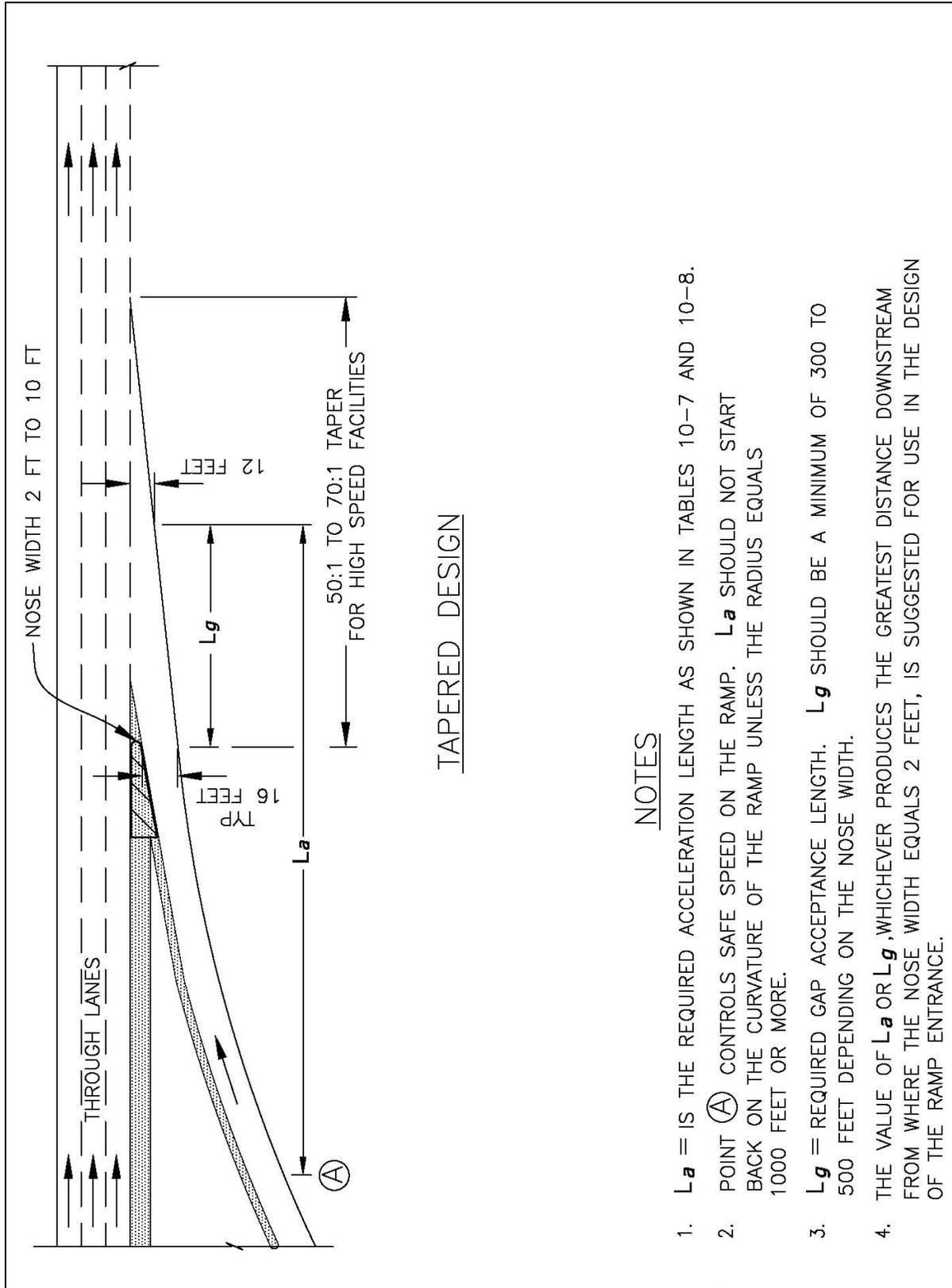


Figure 10-11A [Figure 10-69 of the PGDHS (1)] Typical Single-Lane Entrance Ramps (Tapered)

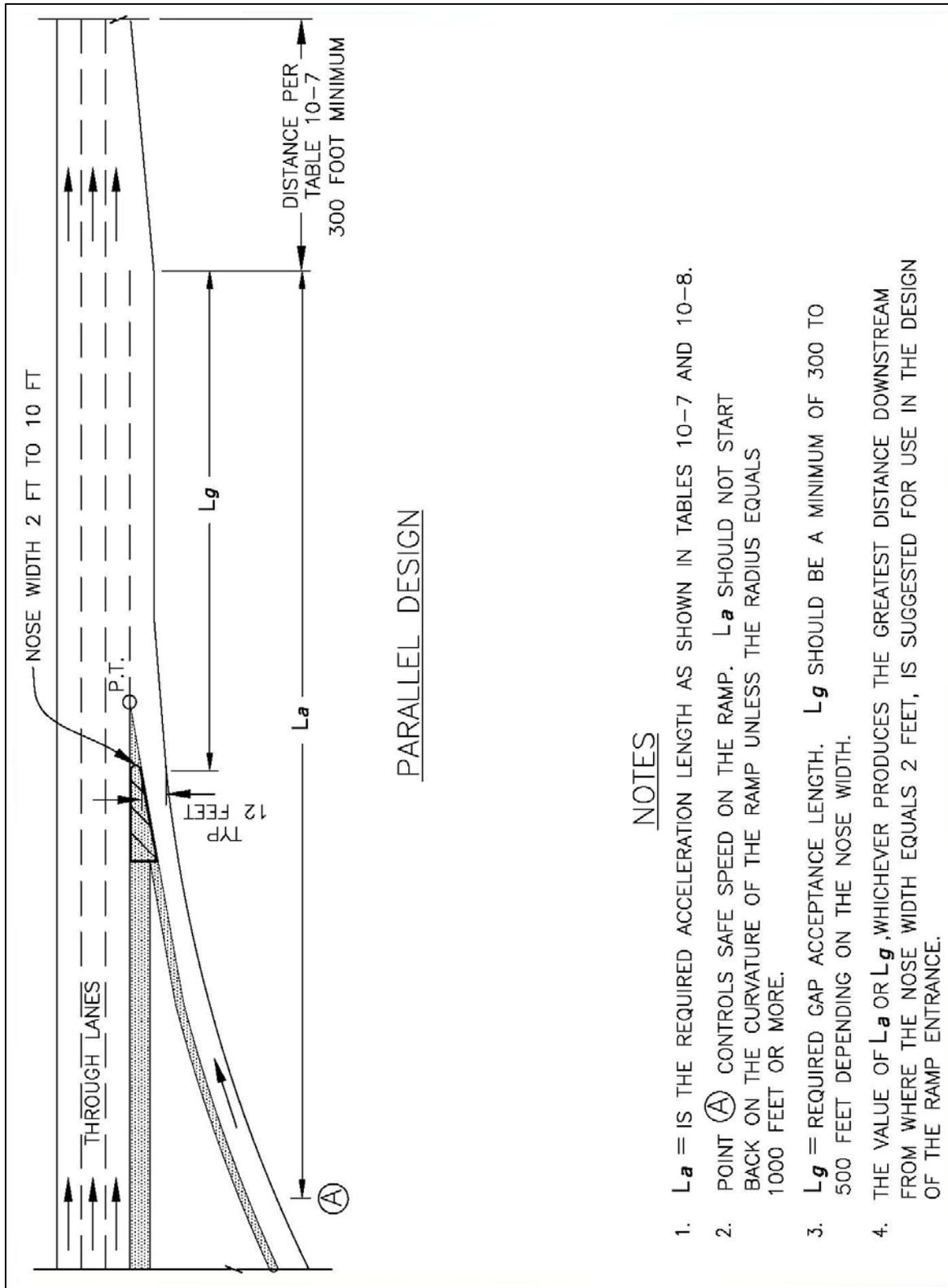


Figure 10-11B [Figure 10-69 of the PGDHS (1)] Typical Single-Lane Entrance Ramps (Parallel) (Preferred)

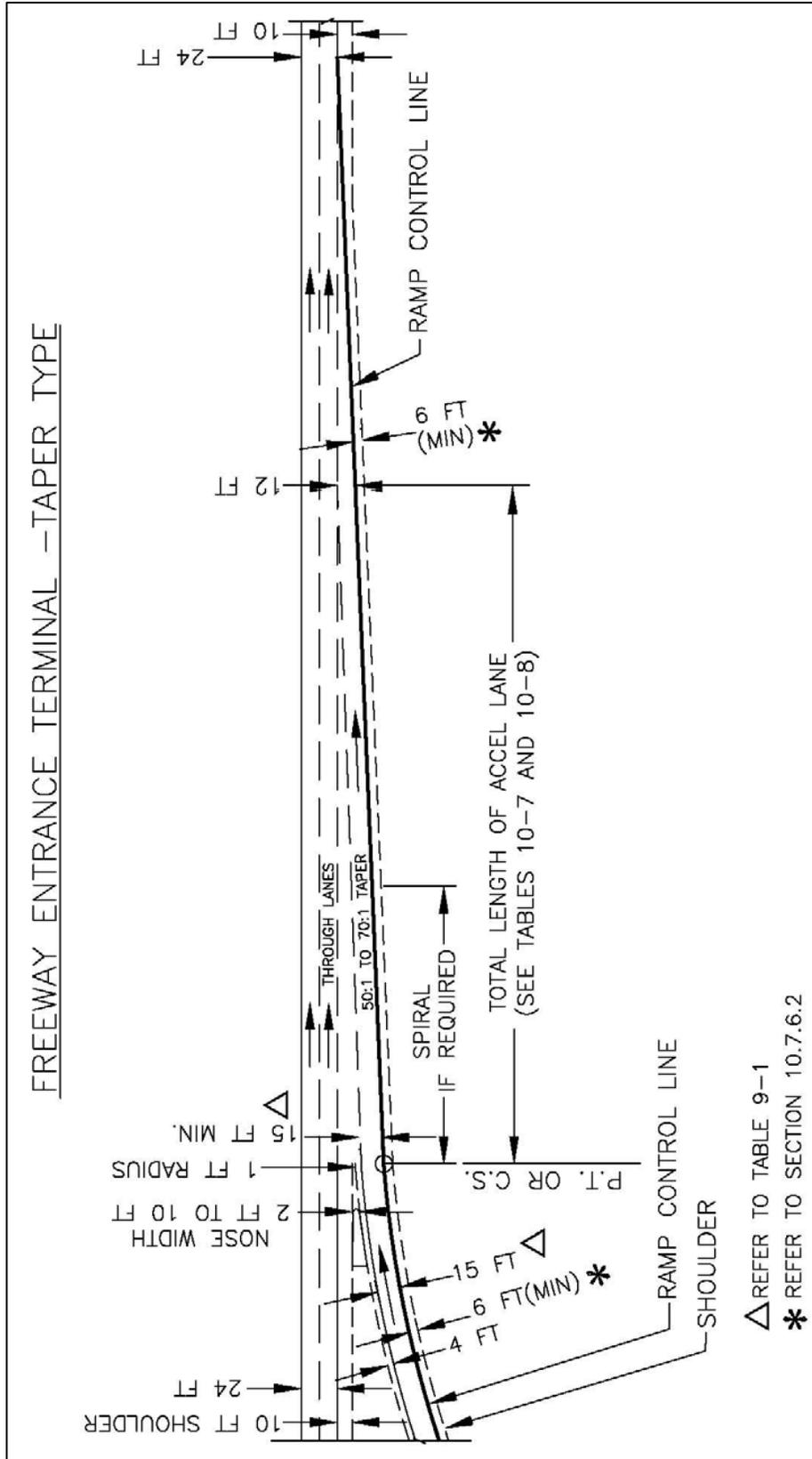
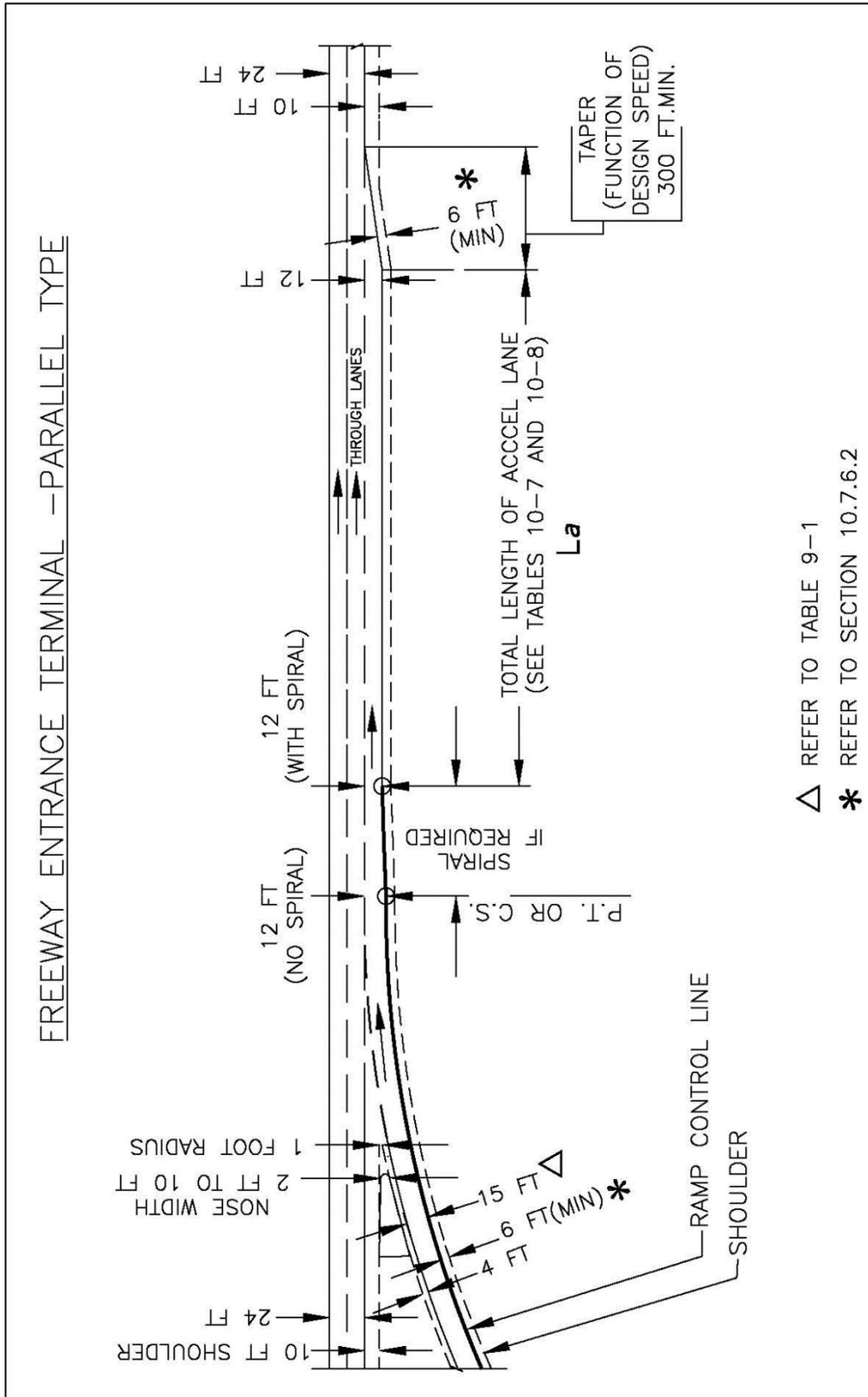


Figure 10-12 Freeway Entrance Terminal – Taper Type



**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.

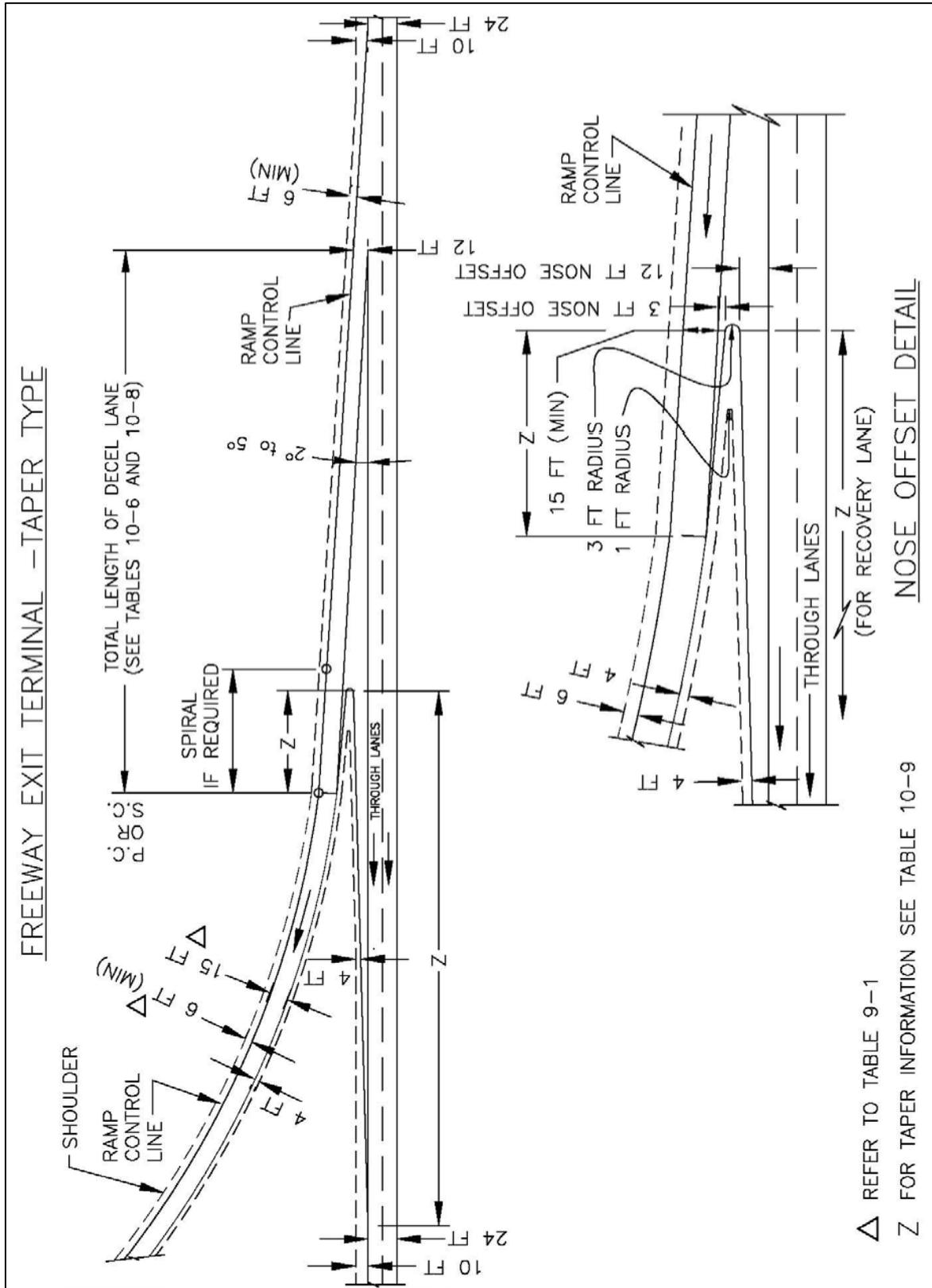


Figure 10-14 Freeway Exit Terminal – Taper Type

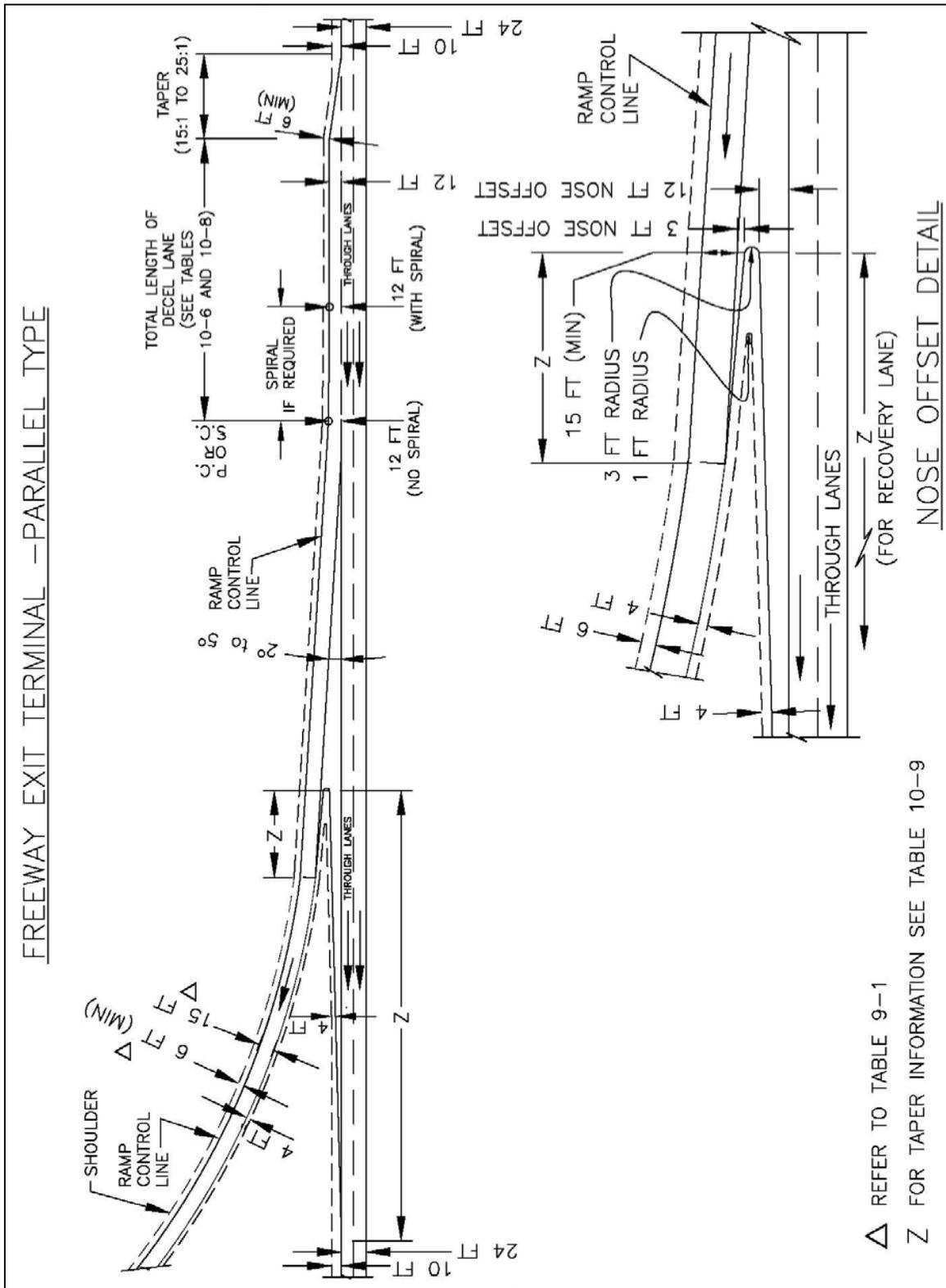


Figure 10-15 Freeway Exit Terminal – Parallel Type

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.

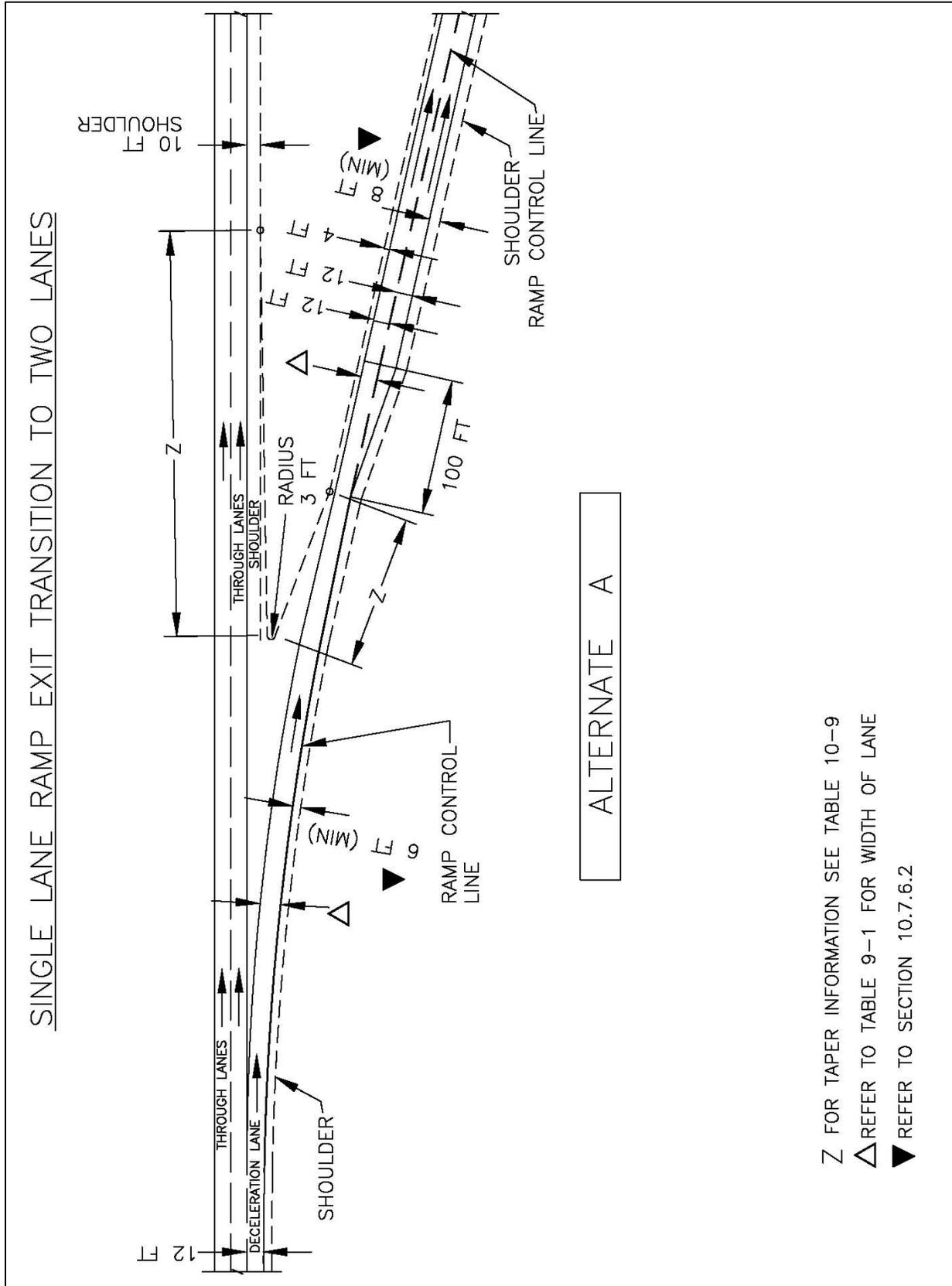


Figure 10-16A Single-Lane Ramp Exit Transition to Two Lanes (Alternate A)

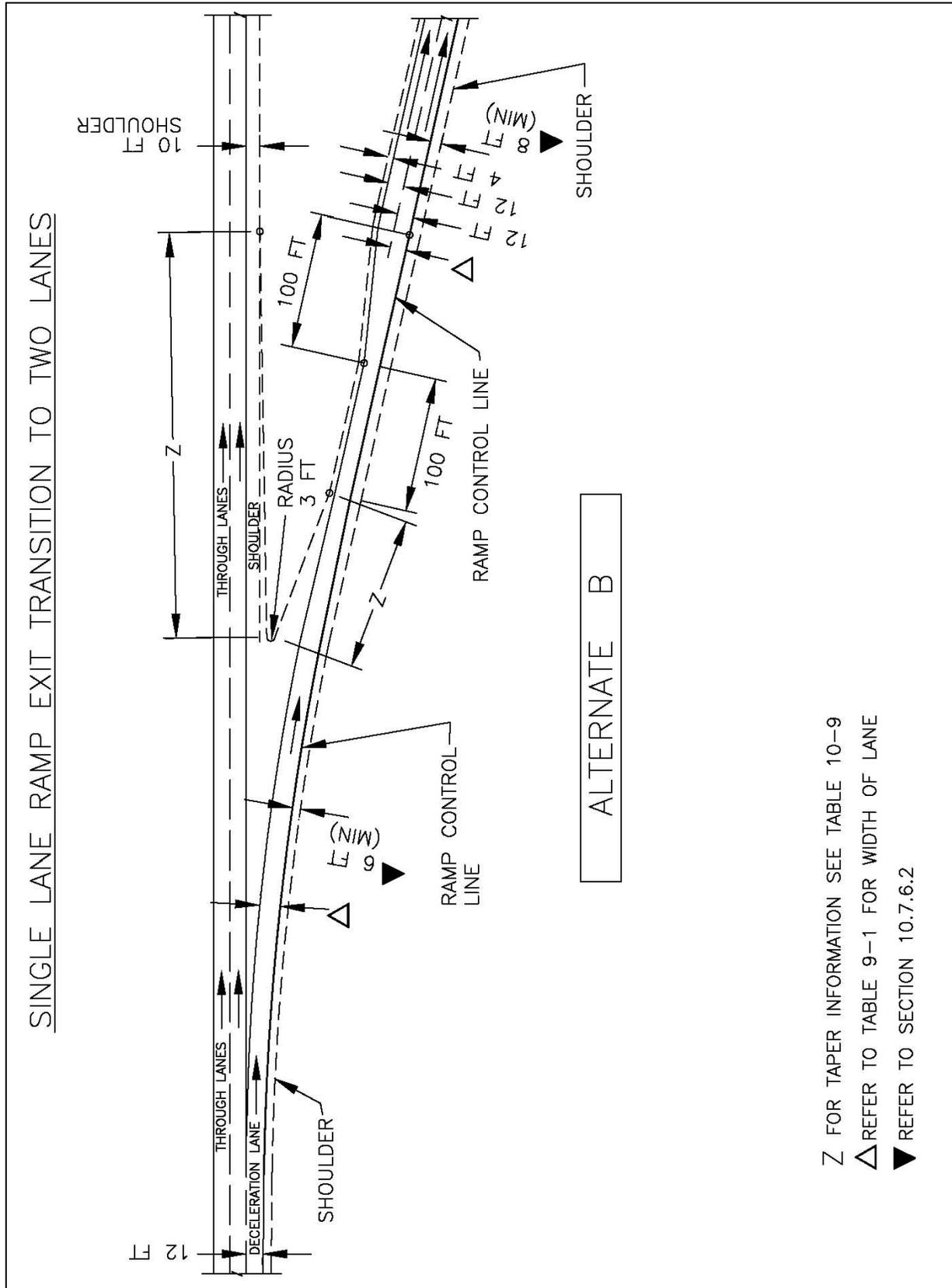


Figure 10-16B Single-Lane Ramp Exit Transition to Two Lanes (Alternate B)

### **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.

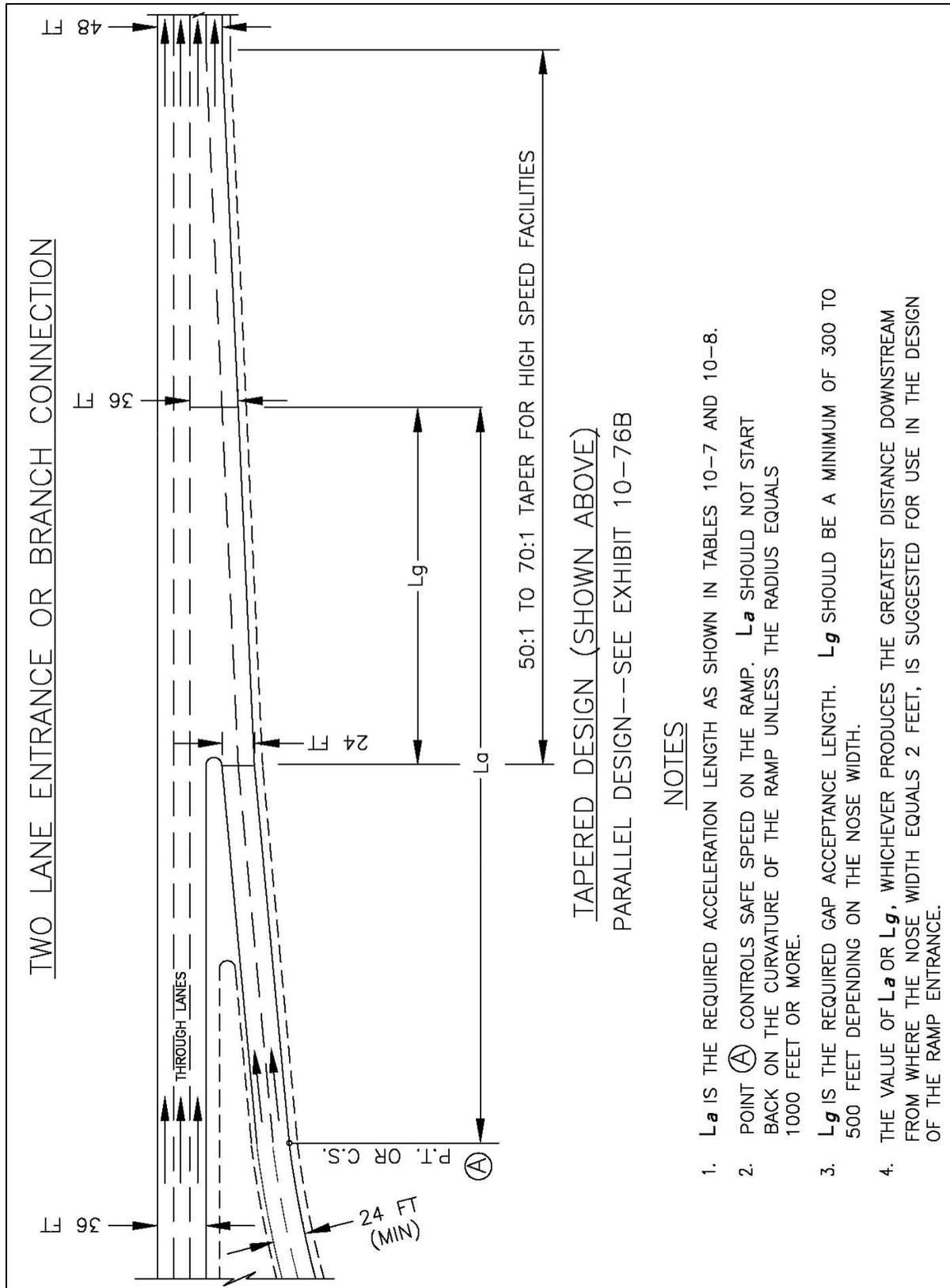


Figure 10-17 Two-Lane Entrance or Branch Connection



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**

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## CHAPTER 12

### ACCESSIBLE PEDESTRIAN DESIGN

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### **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 OPMD 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. OPMDs 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](https://www.fhwa.dot.gov/civilrights/programs/doj_fhwa_ta.cfm)
2. [https://www.fhwa.dot.gov/civilrights/programs/doj\\_fhwa\\_ta\\_glossary.cfm](https://www.fhwa.dot.gov/civilrights/programs/doj_fhwa_ta_glossary.cfm)
3. [https://www.fhwa.dot.gov/civilrights/programs/ada\\_resurfacing\\_qa.cfm](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.

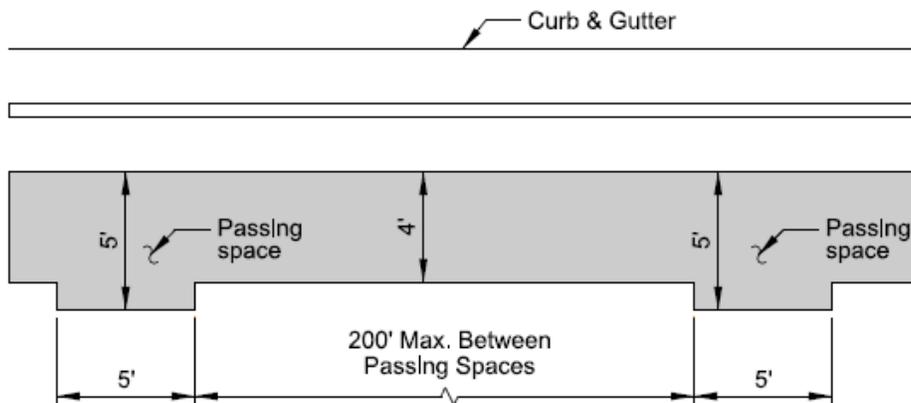
### Pedestrian Access Route Technical Requirements

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.

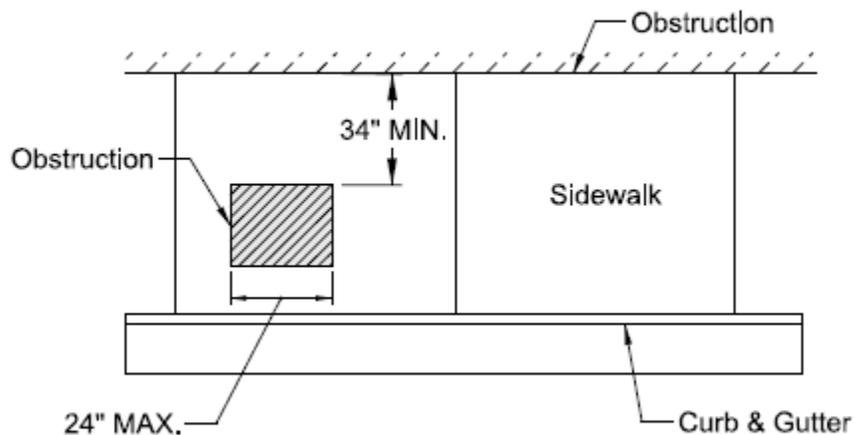
The following describes the common requirements of the PAR.

**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**

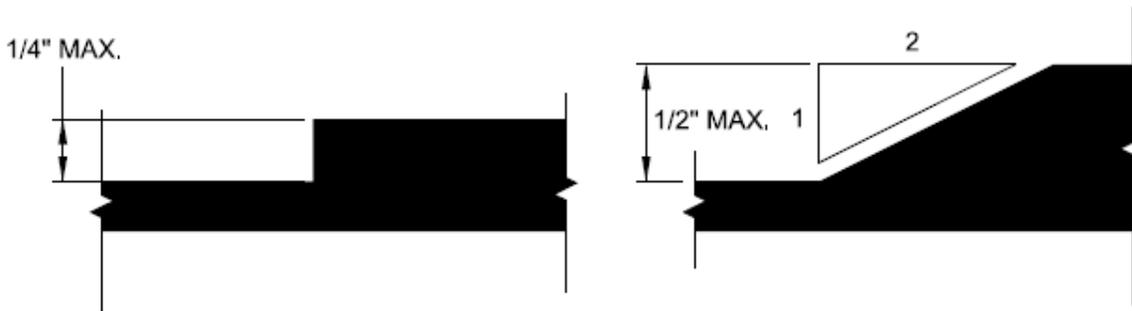
**Pinch Points** - Pinch points within the PAR shall not be less than 34 inches in width and not exceed 24 inches in the direction of pedestrian travel. A study by the U.S. Access Board determined that a 34 inch width allowed 95% of the wheel mobility users to pass the obstruction in the study areas (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.

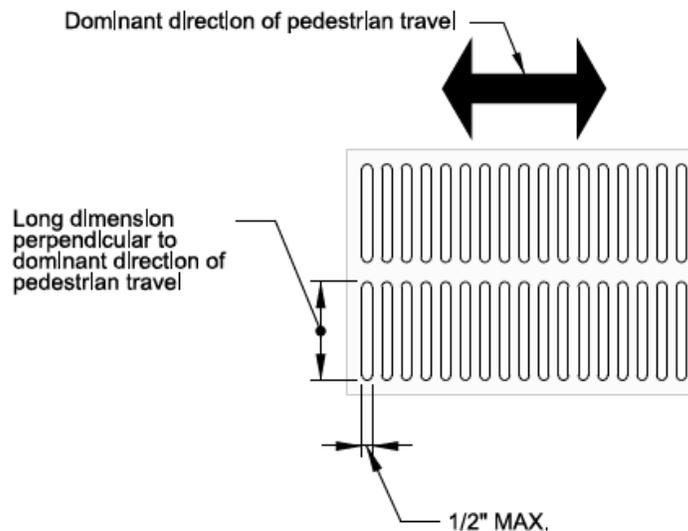
There exists an allowance for vertical surface discontinuities for occasional expansion joints and objects such as utility covers, vault frames, and gratings that cannot be located in another portion of the sidewalk or outside the PAR. Objects such as utility covers, vault frames, and gratings should 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.

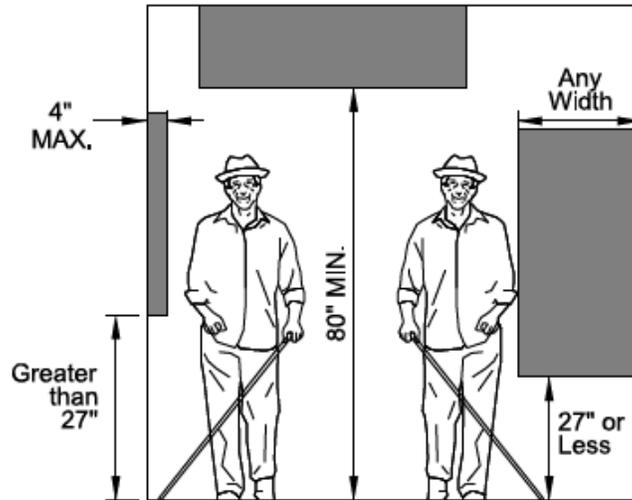
**Horizontal Openings & Horizontal Gaps (R302.7.3)** - The horizontal openings in grates shall not permit passage of a sphere more than 0.5 inch in diameter. Grate openings should be placed so that the long dimension is perpendicular to the direction of travel of pedestrians. The use of grates and inlets within the PAR should be avoided.



**Figure 12-4 - Horizontal Openings in Grates**

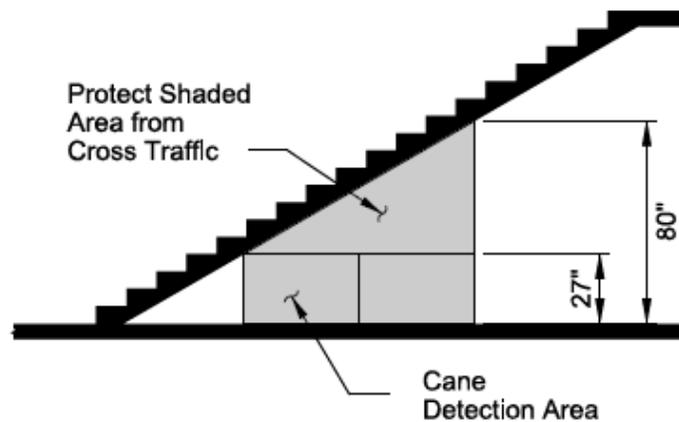
When the PAR crosses a railway the horizontal gap between the pedestrian surface and the rail shall be no more than 2.5 inches on a non-freight railway and no more than 3 inches on a freight railway.

**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.



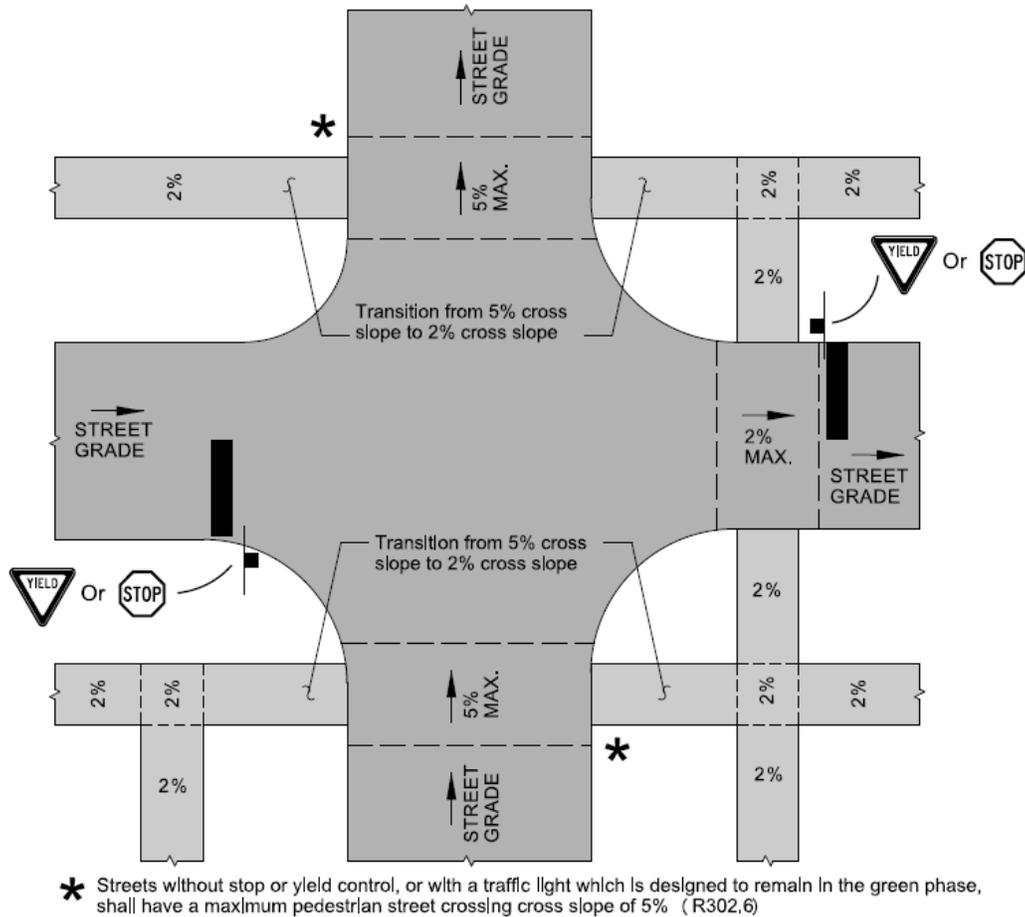
**Figure 12-6 - Protection from Stairways**

**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.

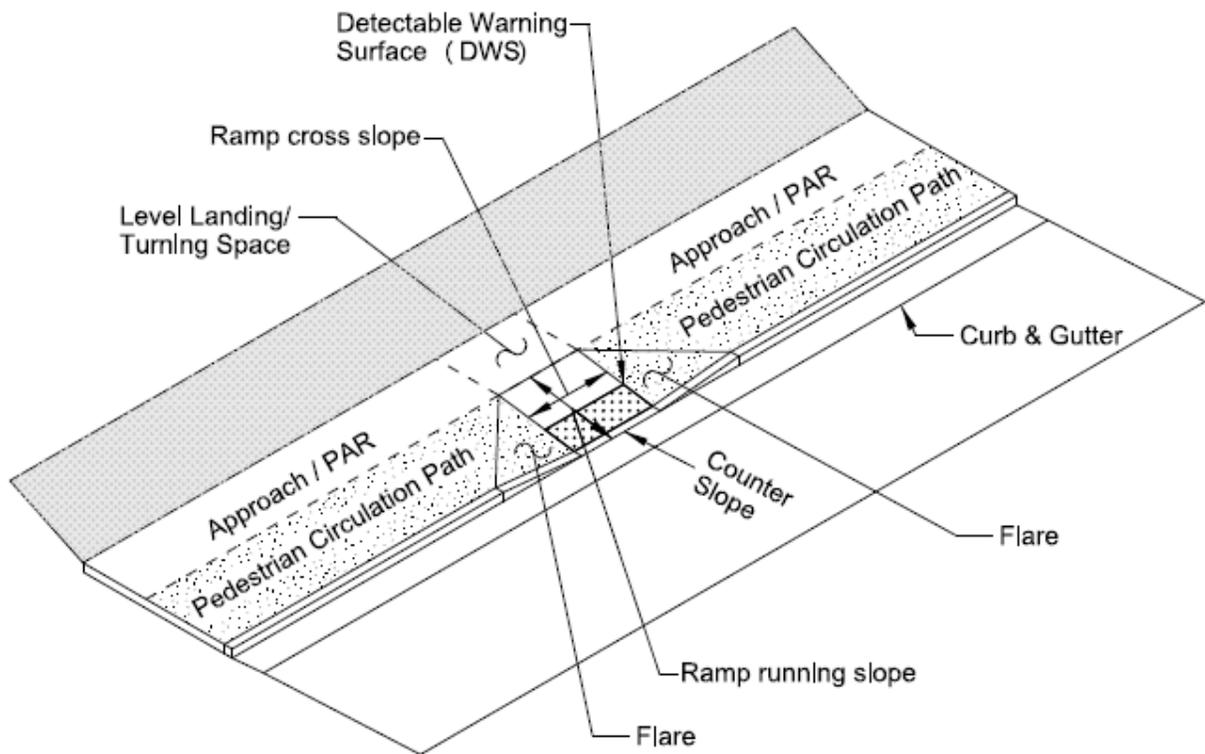


**Figure 12-7 - Pedestrian Street Crossing Cross Slope**

## 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**

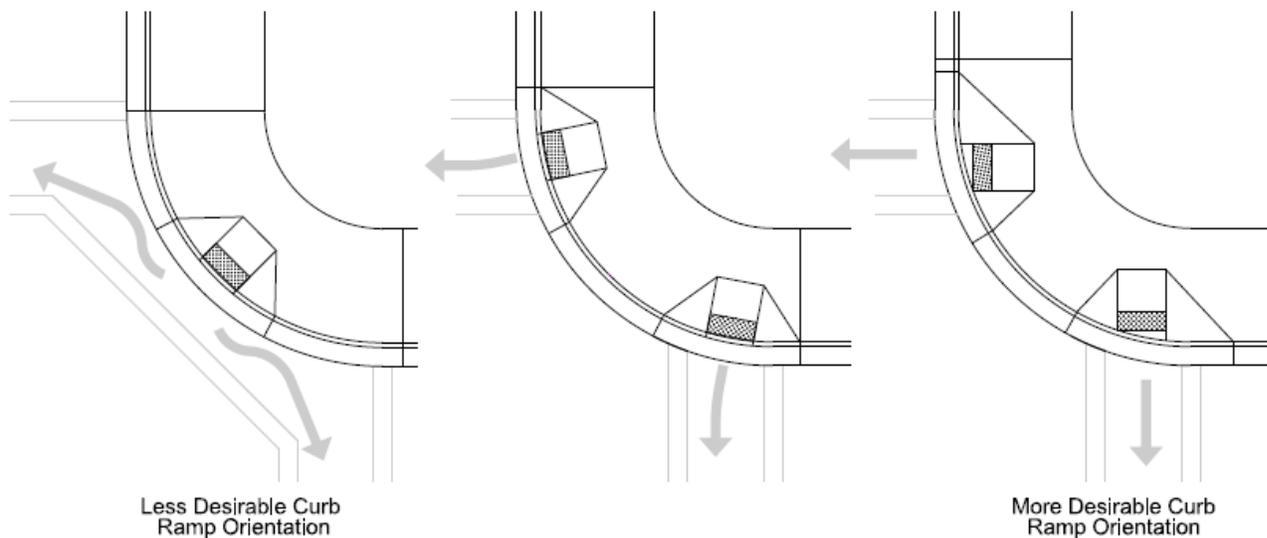
Curb ramps should not be placed in locations where pedestrians must cross drainage inlets or ponding water. Designers should consider the location of drainage inlets relative to any curb ramps that are being installed. Locating or moving drainage inlets upstream of curb ramps helps eliminate ponding 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.

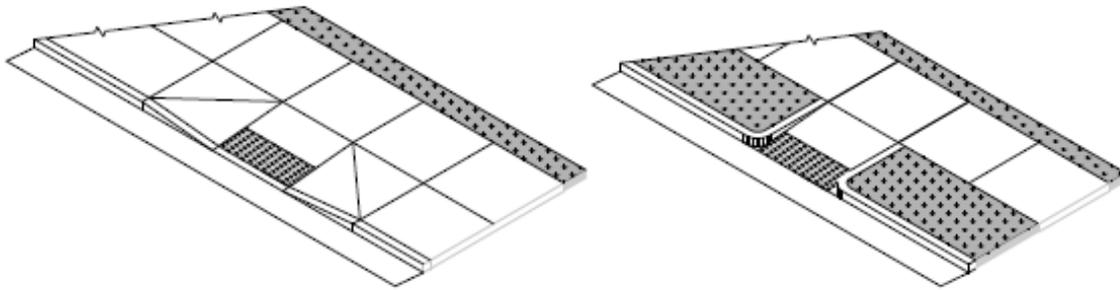


**Figure 12-9 - Preferred Curb Ramp Placement**

## 12.4 CURB RAMP TYPES

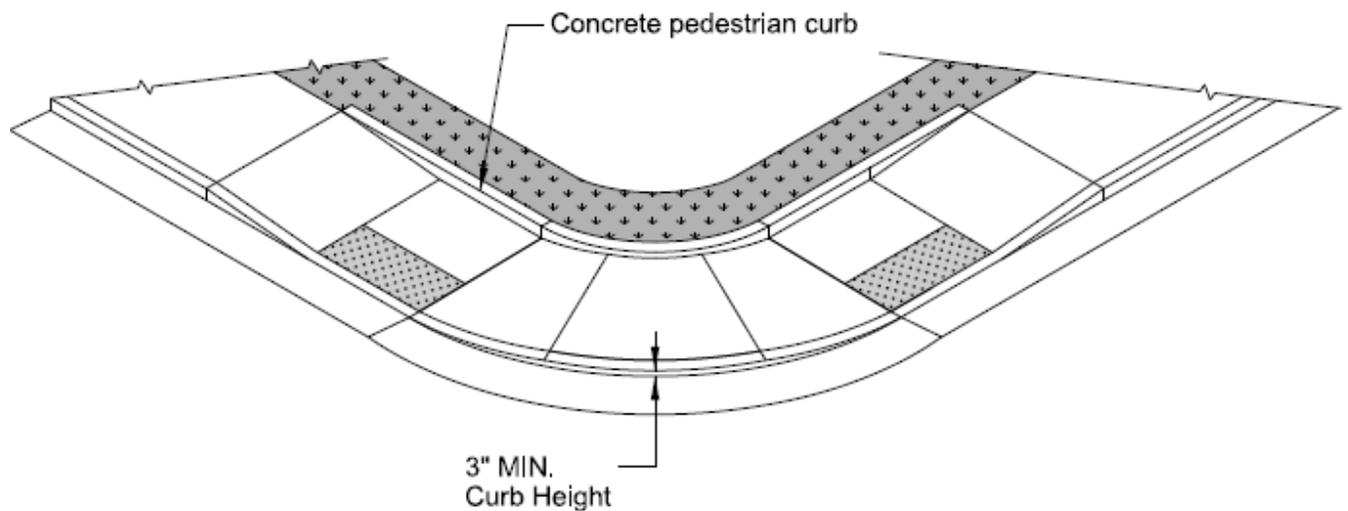
Perpendicular curb ramps are oriented perpendicular to the curb face or vertical elevation change they traverse. Perpendicular ramps have a turning space located at the top of the ramp to allow users to get oriented in the direction of the crossing before travelling down the ramp.

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.



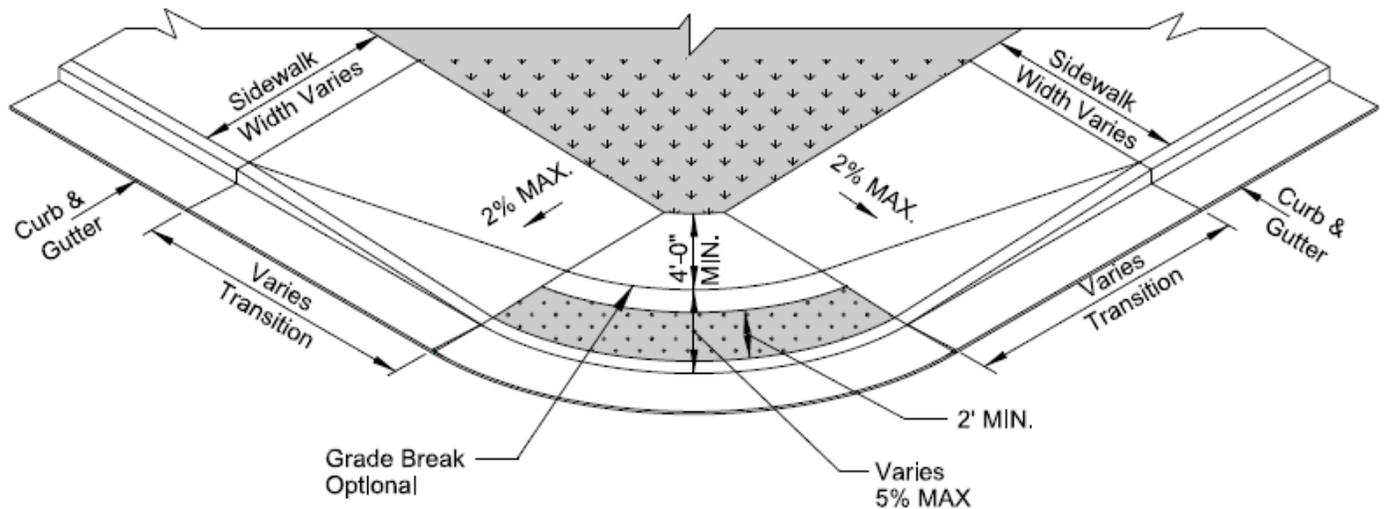
**Figure 12-10 - Perpendicular Curb Ramp Examples**

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:

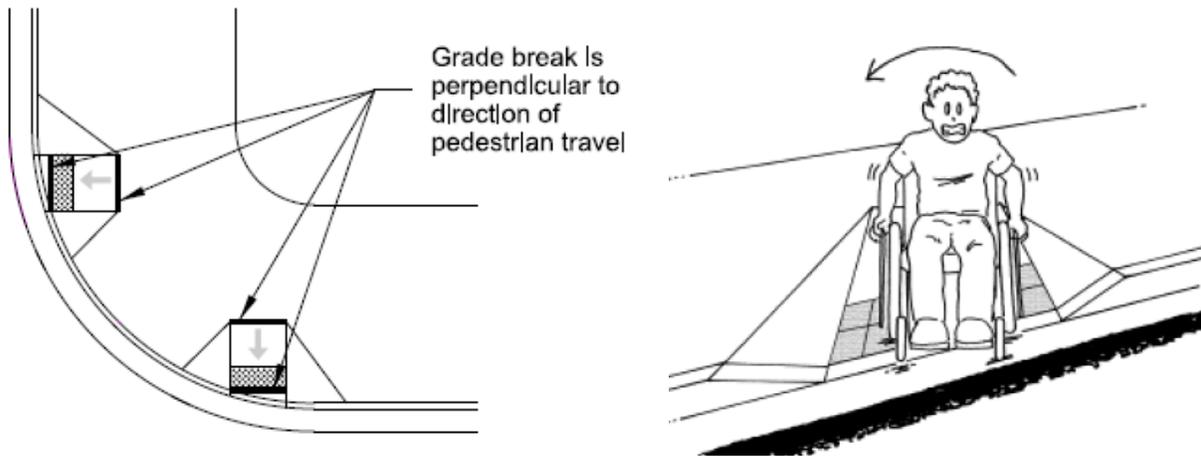
**Ramp Running Slope (R304.2.2)** - Curb ramps shall have a maximum running slope of 8.33%. The running slope of a curb ramp is measured in the center of the ramp run in the direction of pedestrian travel. If the surrounding terrain requires a ramp to chase grade, the ramp is required to be no longer 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 servicing. Curb ramps that service shared-use paths should match the width of the path.

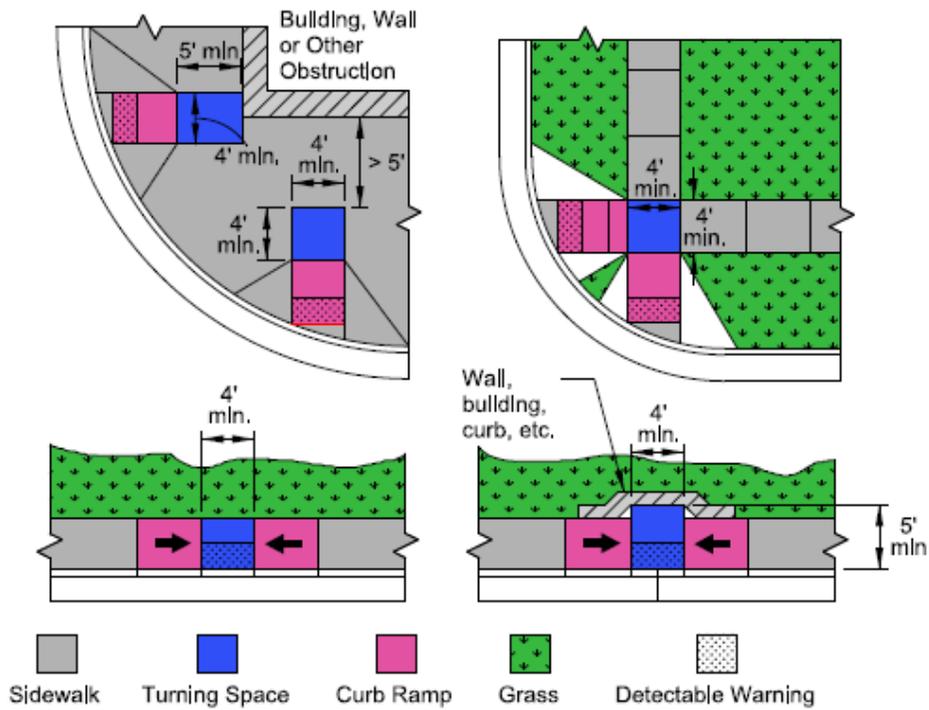
**Grade Breaks (R304.5.2)** - Grade breaks at the top and bottom of curb ramp runs shall be perpendicular to the ramp. Grade breaks are not allowed on the surface of the ramp run or turning space. Surfaces that meet at grade breaks shall be flush. When grade breaks are not

perpendicular to the path of travel they pose challenges to users in wheelchairs because one wheel may strike the ramp before the others.



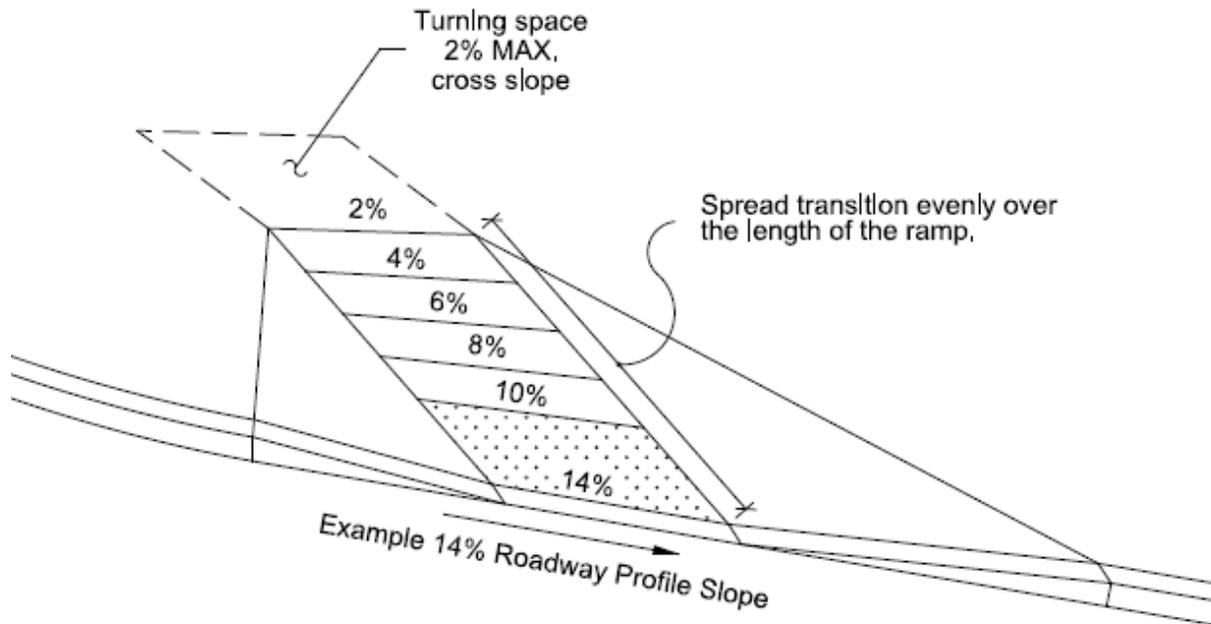
**Figure 12-13 - Curb Ramp Grade Breaks**

**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.



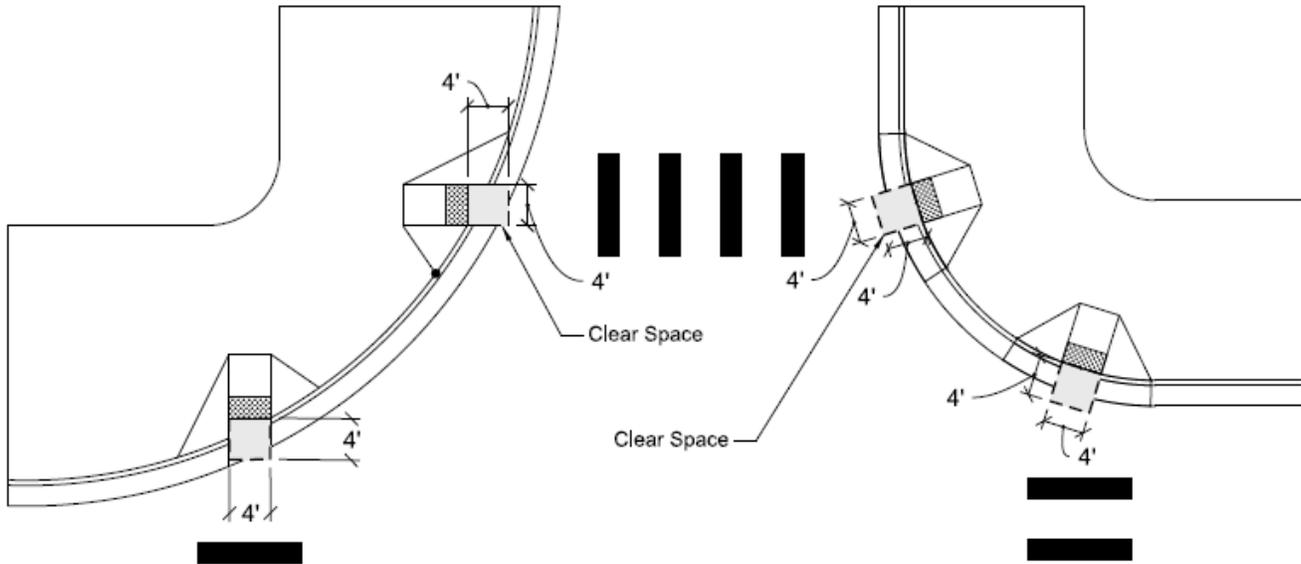
**Figure 12-14 - Curb Ramp Turning Spaces**

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**

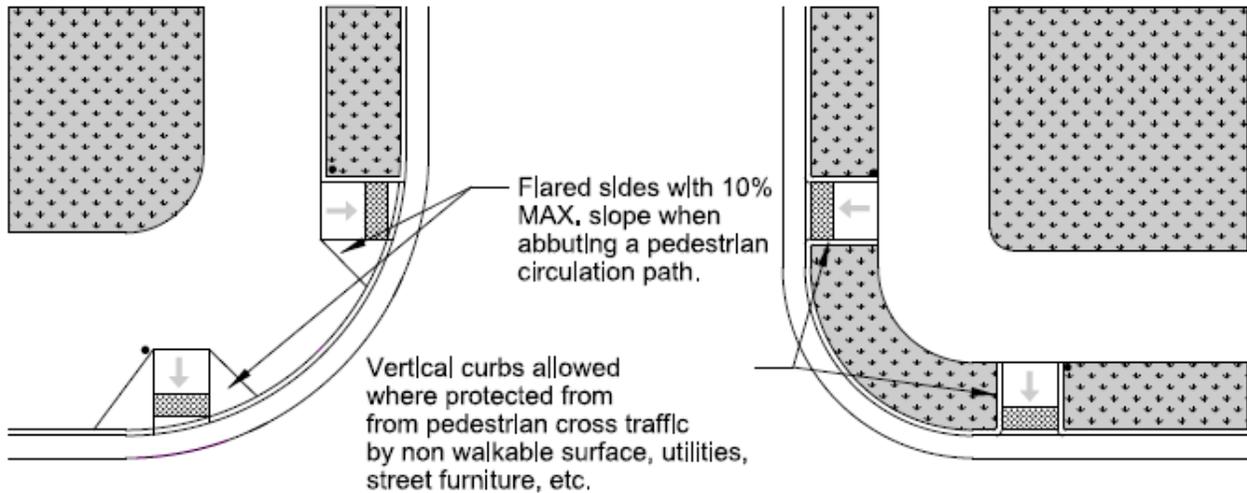
**Clear Spaces (R304.5.5)** - Beyond the bottom grade break of perpendicular ramps, a clear space 4-foot by 4-foot minimum shall be provided. This clear space must be within the pedestrian street crossing (crosswalk) and wholly outside of any vehicle travel lanes.



**Figure 12-16 - Perpendicular Curb Ramp Clear Spaces**

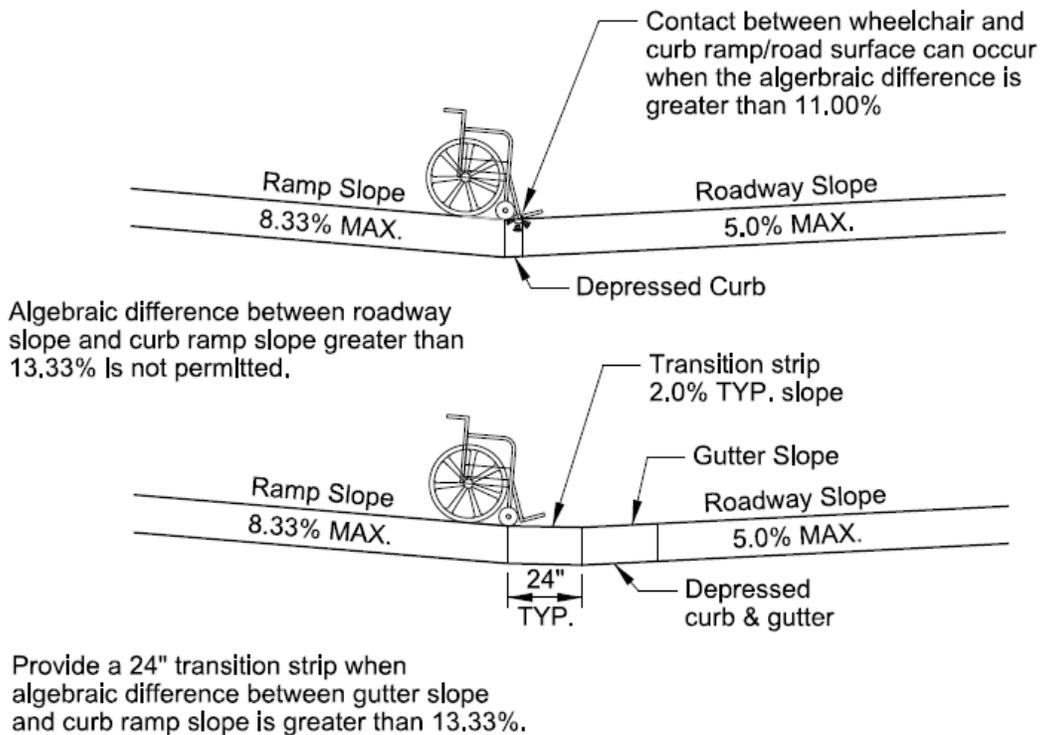
**Flared Sides (R304.2.3)** - Where a pedestrian can walk across a curb ramp, flared sides shall be provided to prevent a tripping hazard. Flared sides shall be sloped at 10.0% maximum and be measured parallel to the curb line. **A best practice is to design flares with slopes between 8.0% and 10.0%**, 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.

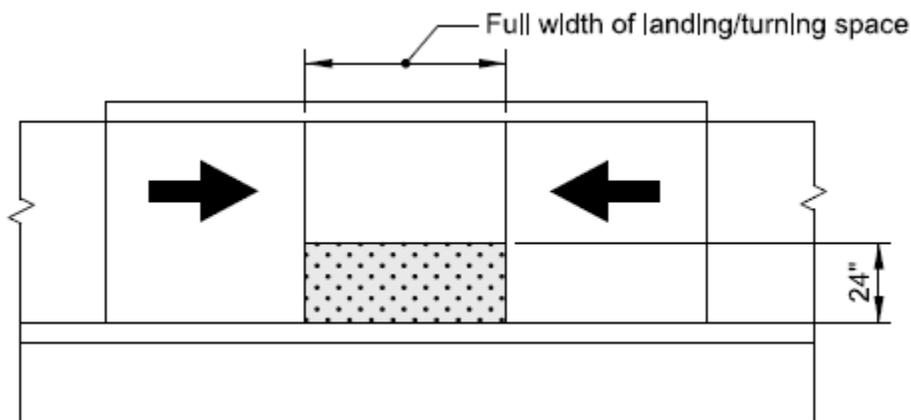
For the technical requirements of the truncated dome size and spacing, see the PROWAG section R305.1 or CDOT Standard Plan M-608-1.

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:

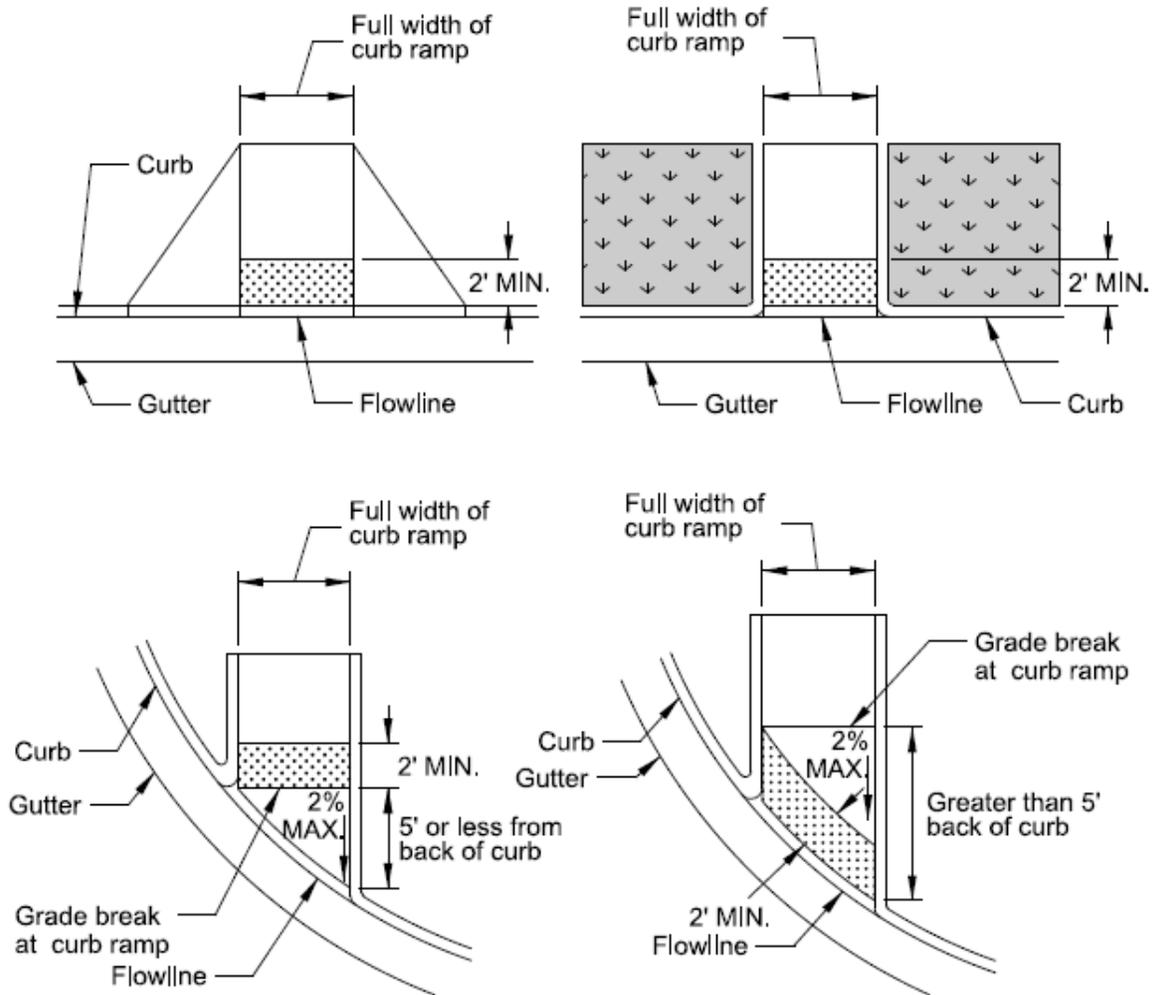
- If there is the presence of a traffic control device that indicates the driveway functions more like a street crossing than a driveway (e.g. lane or pavement markings, signals, stop signs).
- Where a sidewalk descends to meet the grade of the street at the driveway crossing.
- Where the sidewalk changes 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.



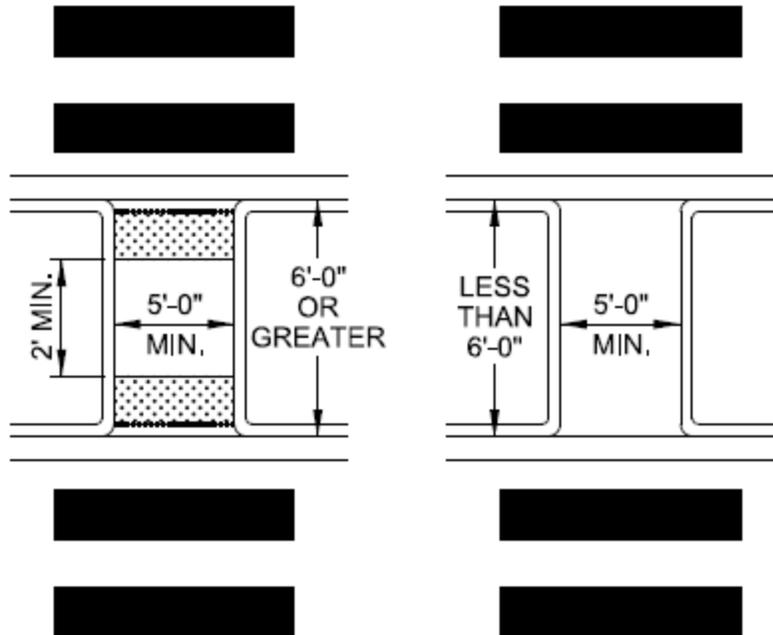
**Figure 12-19 - Parallel Curb Ramp DWS Placement**

*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.

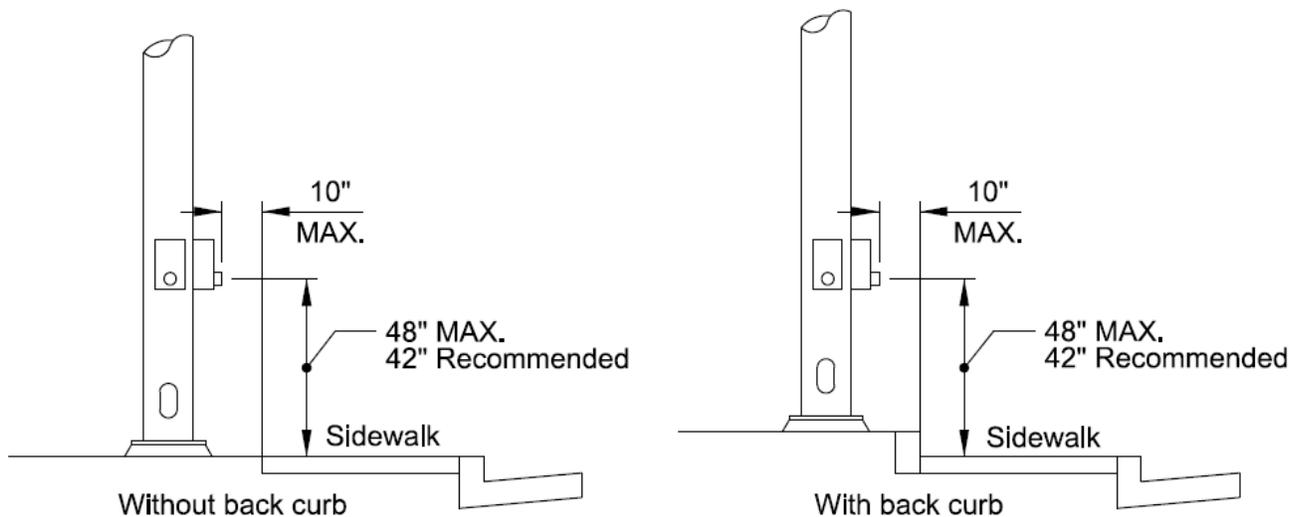
Consistency throughout the pedestrian system is critical. On CDOT signal projects each location should be evaluated to determine if there is a need for APS through the application of CDOT's APS protocol.

[https://www.codot.gov/business/civilrights/ada/assets/cdot\\_aps\\_protocol\\_march\\_2017.pdf](https://www.codot.gov/business/civilrights/ada/assets/cdot_aps_protocol_march_2017.pdf)

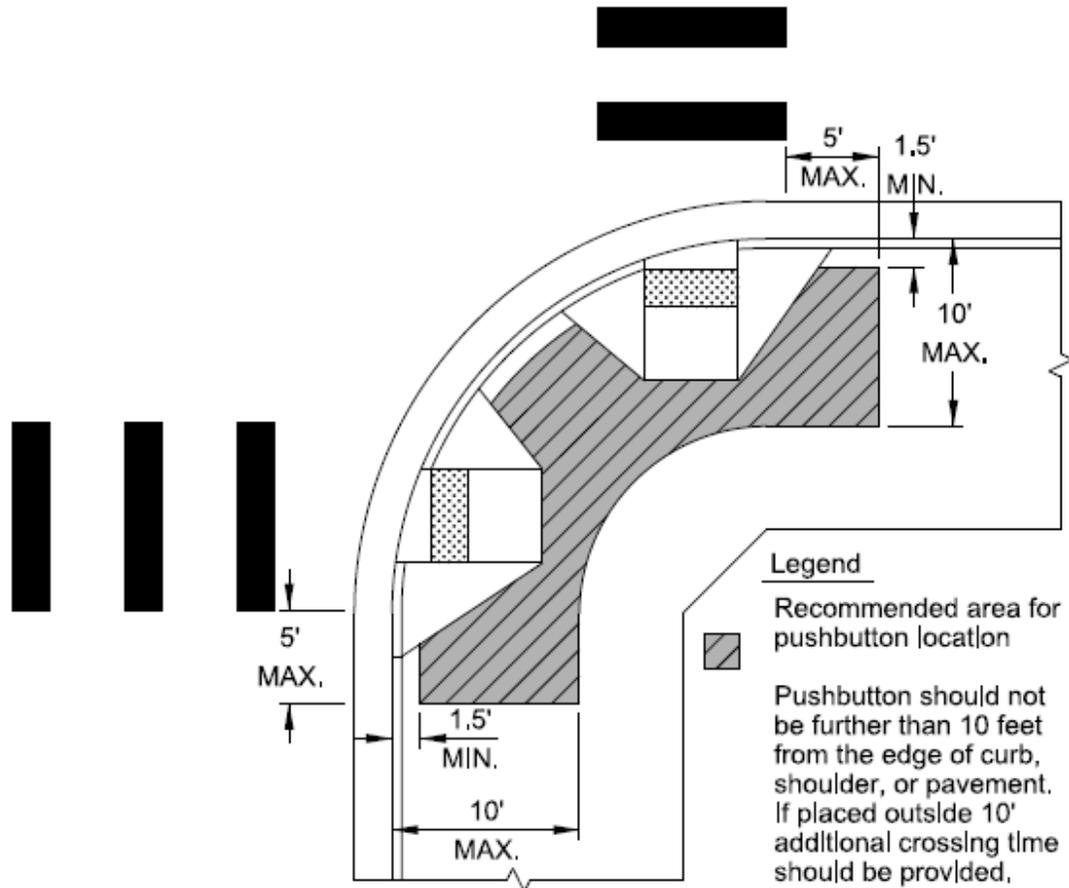
In addition to the requirements in the MUTCD and the PROWAG, the following should be taken into consideration when installing pedestrian crossing controls for pedestrian accessible routes:

- 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**

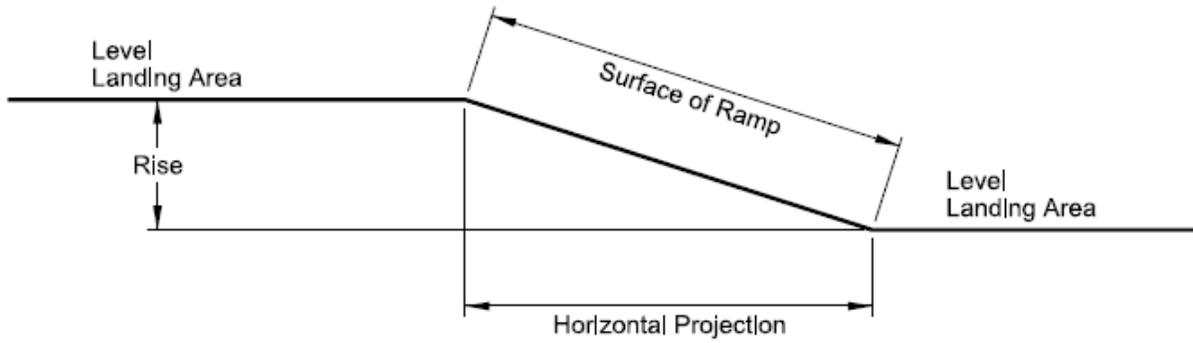


**Figure 12-23 - Ped Push Button Placement**

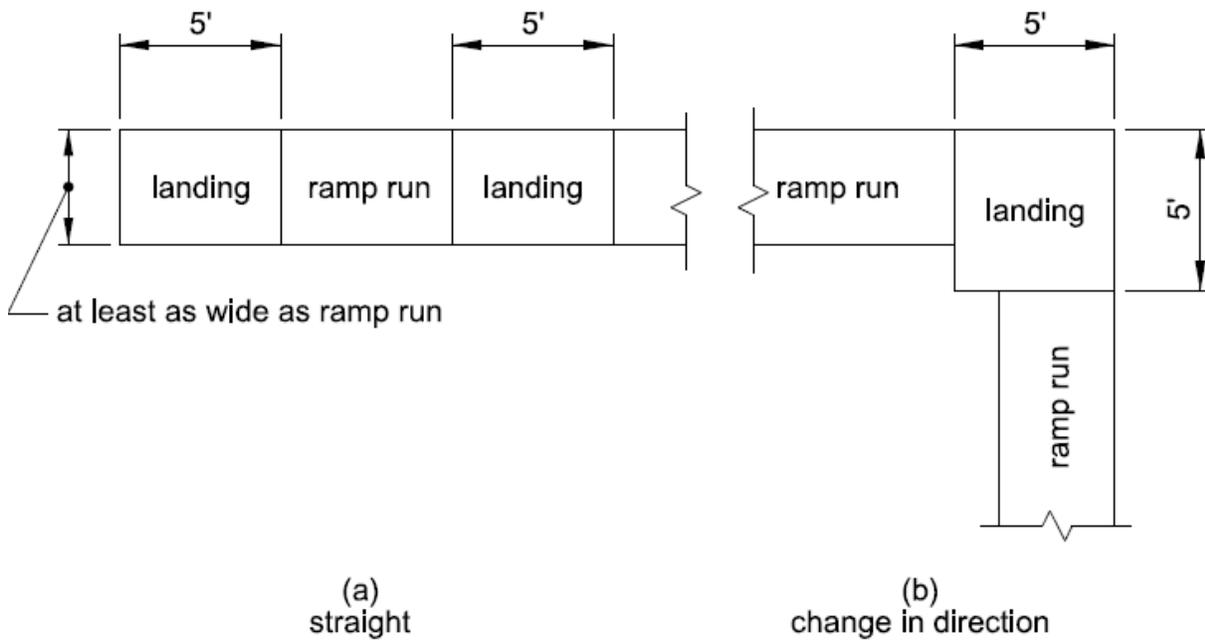
## 12.7 PEDESTRIAN RAMPS & LANDINGS

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**

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## 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.
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## CHAPTER 16

### CONSTRUCTION SPECIFICATIONS

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## **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 citations. 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.



## 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:

<p>REVISION OF SECTION 202 REMOVAL OF ASPHALT MAT</p>	
<p>Section 202 of the Standard Specifications is hereby revised for this project as follows:</p>	
<p>Subsection 202.01 shall include the following:</p>	
<p>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.</p>	
<p>In subsection 202.02 delete the seventh paragraph and replace with the following:</p>	
<p>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:</p>	
<p>(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.</p>	
<p>Subsection 202.11 shall include the following:</p>	
<p>The removal of the existing asphalt mat will be measured by the square yard of mat removed to the required depth and accepted.</p>	
<p>Subsection 202.12 shall include the following:</p>	
<p>Payment will be made under:</p>	
<p><b>Pay Item</b> Removal of Asphalt Mat</p>	<p><b>Pay Unit</b> Square Yard</p>
<p>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.</p>	

- **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:

<b>Pay Item</b>	<b>Pay Unit</b>
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:

<b>Pay Item</b>	<b>Pay Unit</b>
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:

<p>REVISION OF SECTION 210 RESET IMPACT ATTENUATOR</p>	
<p>Section 210 of the Standard Specifications is hereby revised for this project to include the following:</p>	
<p><b>DESCRIPTION</b></p>	
<p>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.</p>	
<p><b>MATERIALS</b></p>	
<p>The impact attenuators are the types shown at the various locations on the plans.</p>	
<p><b>CONSTRUCTION REQUIREMENTS</b></p>	
<p>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.</p>	
<p>The impact attenuator shall be installed in accordance with the manufacturer's recommendations.</p>	
<p><b>METHOD OF MEASUREMENT</b></p>	
<p>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.</p>	
<p><b>BASIS OF PAYMENT</b></p>	
<p>The accepted quantities will be paid for at the contract unit price for the pay item listed below.</p>	
<p>Payment will be made under:</p>	
<b>Pay Item</b>	<b>Pay Unit</b>
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](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 to 10 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 to 10 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 <sup>st</sup> , 5 <sup>th</sup> , 20 <sup>th</sup>	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](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>]

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**TRAFFIC AND SAFETY ENGINEERING**

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## **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.

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