Chapter 10
Grade Separations and Interchanges

Legend

	Multimodal Application Example
	Context-Sensitive Solutions Application Example
	Performance-Based Practical Design Application Example
	Multimodal (MM)
	Context-Sensitive Solutions (CSS)
	Performance-Based Practical Design (PBPD)
	Web link for additional information
	AASHTO-Specific Information

Chapter

10

10 Grade Separations and Interchanges

10.1 Introduction

Grade separations and interchanges are most commonly associated with freeways. As defined in Chapter 5 of this Guide, a Freeway is "free" of at-grade intersections. Nevertheless, at-grade intersections are still elements of consideration within the specialty of grade separation and interchange design. For example, interchange design typically also involves at-grade intersection design at ramp termini. Furthermore, at-grade intersections are also typically an integral part of the larger transportation network.

A transportation network influenced by many factors, including the design characteristics of one interchange, with its own effects reaching far beyond one interchange often spilling into adjacent at-grade intersections. Accommodating high volumes of traffic safely and efficiently through an intersection depends largely on how the intersecting traffic lanes are configured. Grade separations of intersecting traffic lanes facilitate uninterrupted flow or traffic, resulting in the greatest efficiency, safety, and capacity for traffic and the least amount of air pollution.

An interchange is a system of two or more interconnecting roadways or highways and one or more grade separations. Interchange design is the most specialized and highly developed form of intersection design. The designer should be thoroughly familiar with the following relevant material in Chapter 8 of this Guide and apply it to interchange design:

- General factors affecting design.
- Basic data required.
- Principles of channelization.
- Design procedure.
- Design standards.

An interchange or series of interchanges on a freeway or expressway adjacent to or through a community may have a large economic effect on large contiguous areas or even the entire community. Interchanges must be located and designed to provide the most desirable overall plan of access, traffic service, and community development. For this reason, it is recommended to have an active public process to gain input for the development of context sensitive solutions.

The type and design of a grade separation and interchange are influenced by factors such as highway classification, contextual classification, character and composition of traffic, design speed, and degree of access control. The consideration of these controls, plus signing requirements, economics, terrain, environment, and right of way, result in a design with adequate capacity to safely accommodate the traffic demands. The design of an interchange must be considered in conjunction with adjacent interchanges, driver expectancy, and the at-grade intersections on the intersecting corridors.

Interchange types are characterized by the basic shapes of ramps, for example, diamond, loop, directional, "urban," and cloverleaf interchanges. Figure 10-1A, Figure 10-1B, Figure 10-1C, Figure 10-1D, and Figure 10-1E illustrate these basic interchange types. Cloverleaf interchanges usually have a larger footprint and require more right of way and easements than other interchange types. Cloverleaf interchanges are not as common anymore due to more operationally efficient interchange alternatives that do not require as large of a physical footprint.

Alternative types of intersections and interchanges, such as diverging diamond interchange (DDI), continuous flow intersection (CFI) (displaced left turn), and single point urban interchange (SPUI), have modified intersection turning movements that improve traffic operations for high travel demands. These configurations can be implemented as local street intersections or as part of freeway-to-freeway interchanges.

For further information on grade separations and interchanges, refer to Chapter 10 of the American Association of State Highway and Transportation Officials (AASHTO) *A Policy on Geometric Design of Highways and Streets* (the Green Book) (2018 AASHTO GDHS) (AASHTO, 2018).

Figure 10-1A Interchange Types

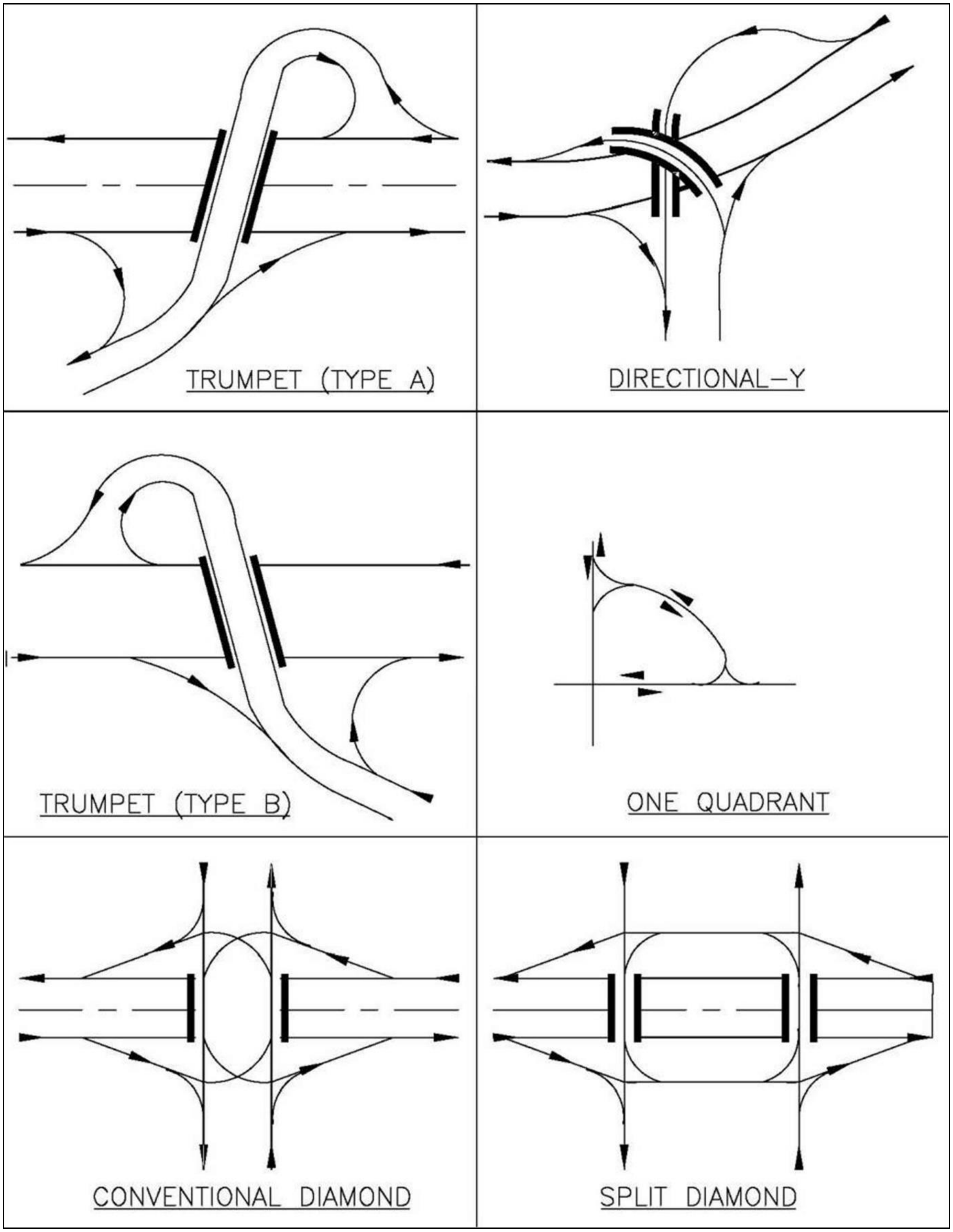


Figure 10-1B Interchange Types

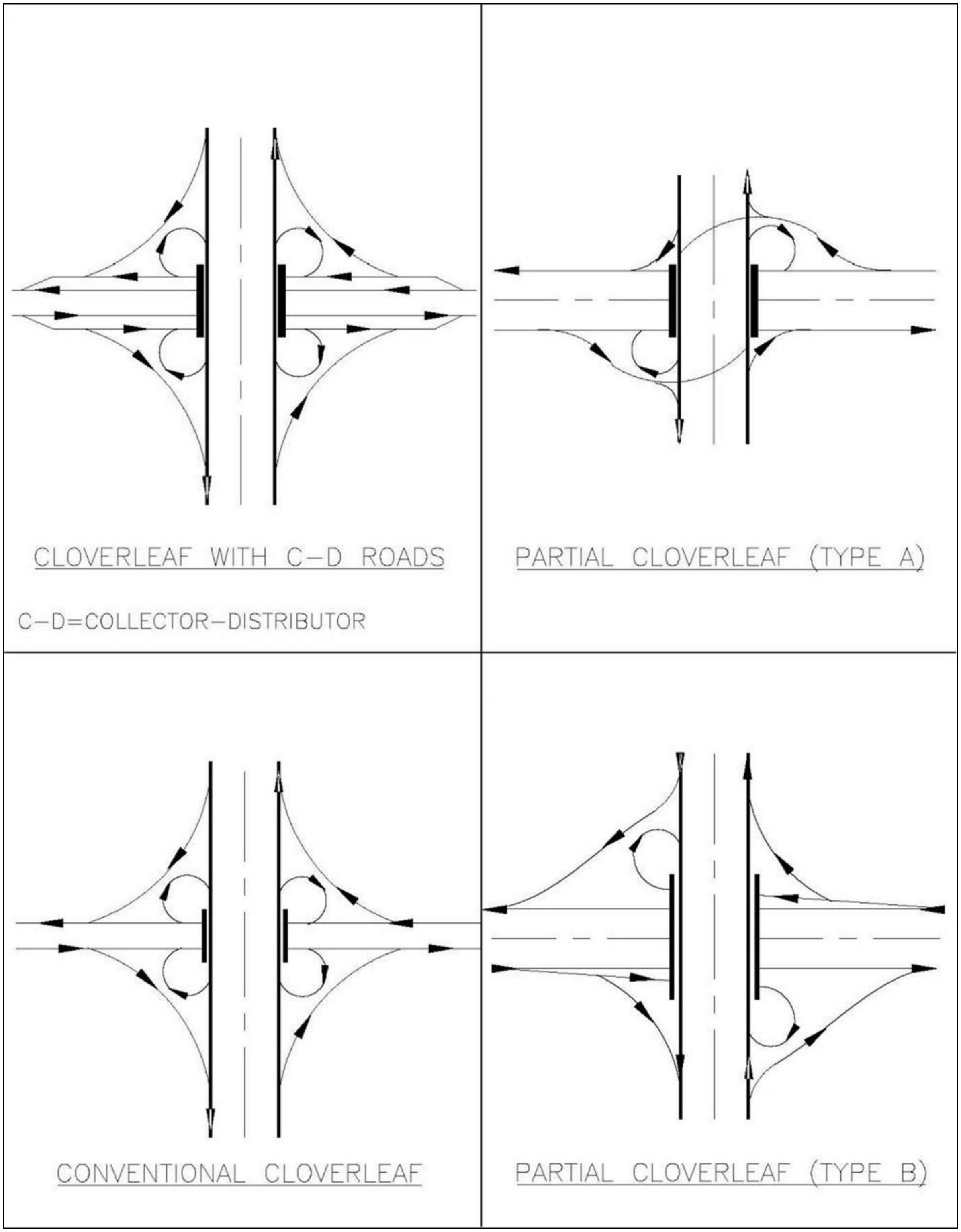


Figure 10-1C Interchange Types

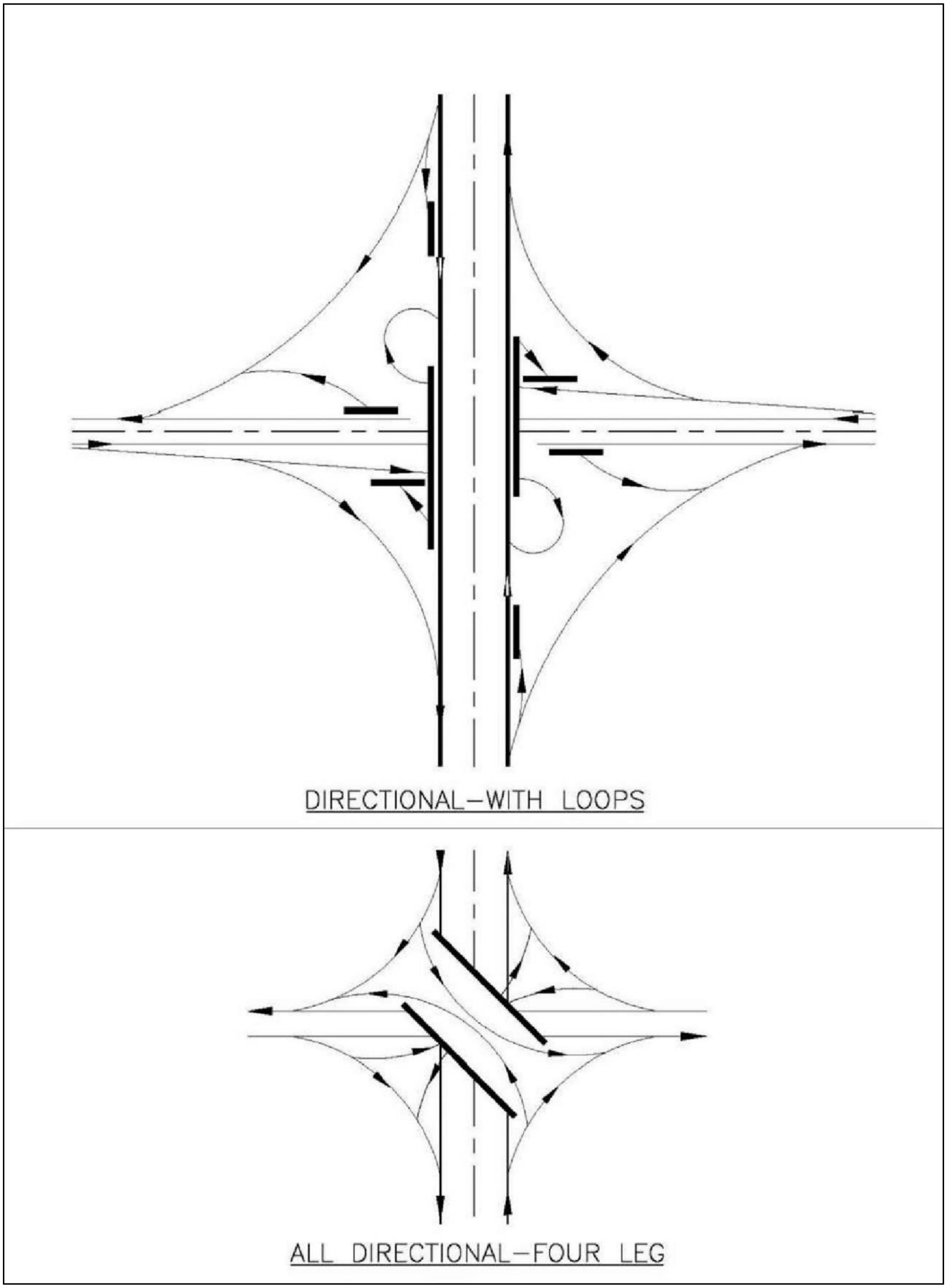


Figure 10-1D Interchange Types

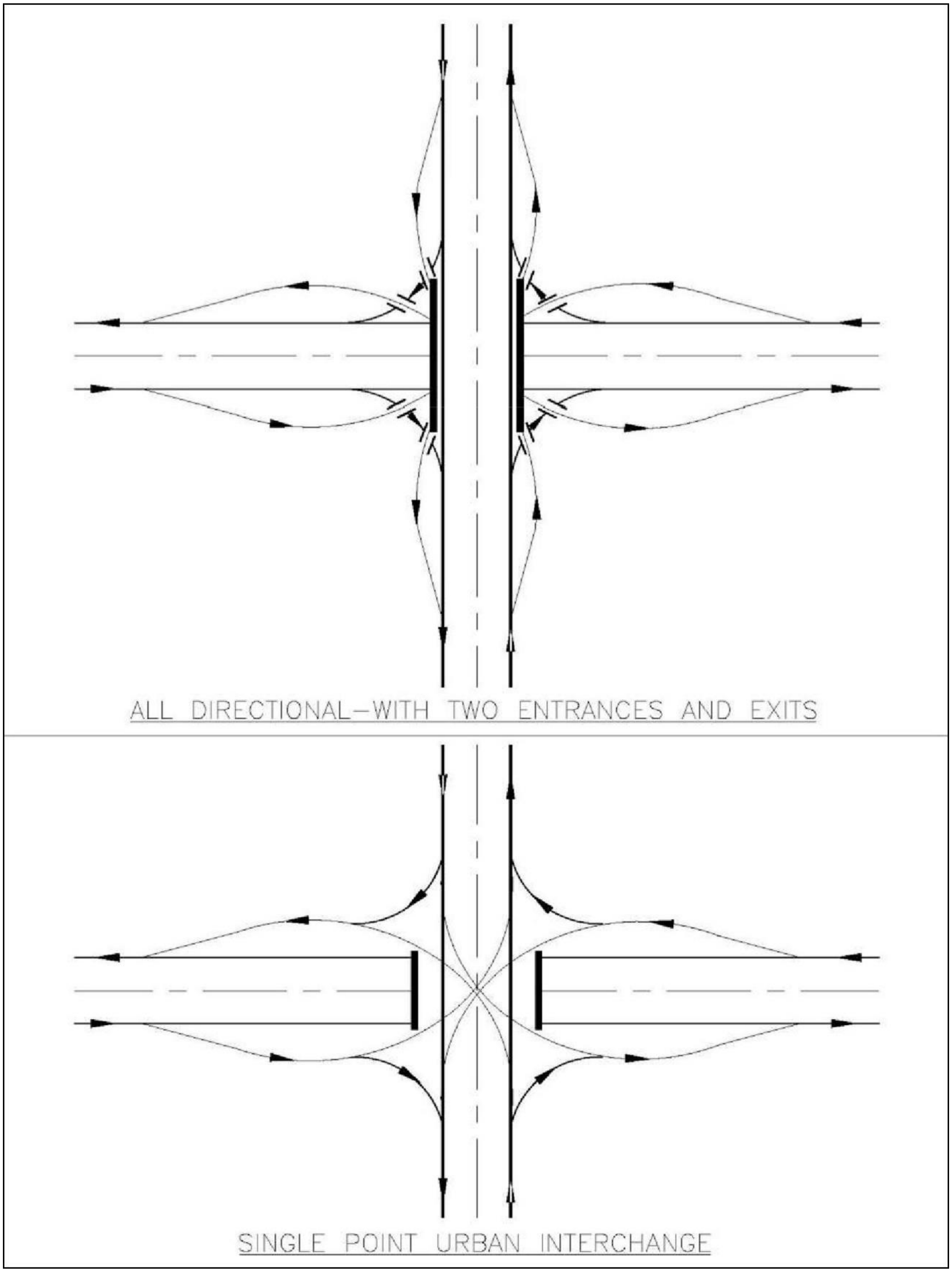
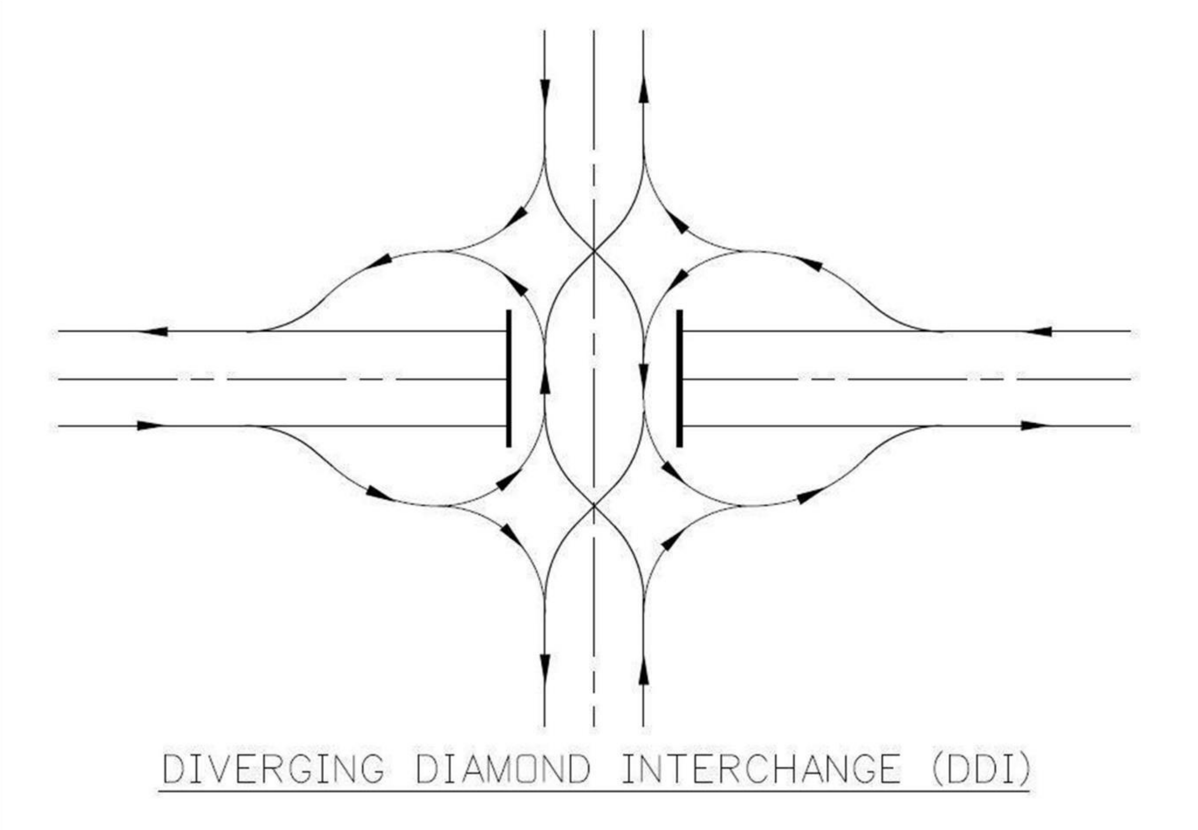


Figure 10-1E Interchange Types



10.2 Warrants for Interchanges and Grade Separations

Intersecting interstates or freeways generally warrant an interchange. A grade separation or interchange also may be warranted at intersecting rural freeways, non-interstate highways, expressways, or 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 and data driven analysis. CDOT Policy Directives 1601.0 Interchange Approval Process (CDOT, 2021) and Procedural Directive 1601.1 Requests for Interchange Access and Modifications to Existing Interchanges on the State Highway System (CDOT, 2022) must be followed.

When determining conditions that may warrant an interchange, the following should be considered:

- *Design designation.* Determining if a corridor warrants a freeway designation and improvements to interchanges along that corridor.
- *Reduction of bottlenecks or intersection congestion.*

- *Reduction of crash frequency and severity. At intersections with a disproportionate frequency or severity of crashes, a grade separation* or interchange may be warranted if inexpensive methods of crash reduction are likely to be ineffective or impractical. Higher crash frequencies can occur at intersections of comparatively lightly traveled highways in rural areas where speeds are high and at heavily traveled intersections.
- *Improved operational efficiency for all traffic movements.*
- *Site topography.* The site topography and the grades of the intersecting roadways help to determine the 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.* Reduction in travel time delay and improvements in travel time reliability.
- *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.* Improved transit operations and transit time reliability.
- *Functional classification of the road.*
- *Transportation Demand Management (TDM) strategies.* These preserve the long-term functionality of the constructed interchange improvements and preserve the overall functionality and operability of the highway system. Updated TDM plans are also required with Type 1 and 2 interchange modifications requests under CDOT Policy Directive 1601.1 Requests for Interchange Access and Modifications to Existing Interchanges on the State Highway System (CDOT, 2022).

10.3 Adaptability of Highway Grade Separations and Interchanges

The three types of intersections are:

- At-grade intersections.
- Highway grade separations without ramps.
- Interchanges.

Factors that would determine the need for a grade separation or interchange and type include:

- *Traffic and Operations.*
- *Context Classification.*
- *Site Conditions.*
- *Design Speed.* 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. 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.* Evaluating the current crash patterns and rates to determine if the addition of an interchange can reduce the crash rates by removing specific crash types at the intersection.
- *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, right of way acquisition, grading and landscaping of large areas.
- *Maintenance Costs.* 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.

10.3.1 Traffic Operations

A traffic operations analysis validates the need for a grade separation or interchange. During initial project scoping, many times during the NEPA phase of a project, traffic analysis can help identify specific operations criteria that need to be met for a grade separation or an interchange to be warranted. The analysis evaluates the current conditions of the intersection to determine the operational efficiency or Level of Service (LOS) for the intersection and the individual legs of the intersection. Refer to Chapter 8 of this Guide regarding how to determine the intersection LOS.

10.3.2 Site Conditions

Site conditions may justify the need for a grade separation or interchange, for example, a grade separation over or under a rail line, access to properties or areas where no access currently exists, or a crossing to serve a high volume of pedestrian or bicycle traffic.

10.3.3 Type of Highway and Intersecting Facility

The functional type of the intersection highway or roadway dictates the required type of grade separation or interchange. An interchange at intersecting interstates or freeways must be designed to maintain an uninterrupted flow of traffic. Arterials feeding to the interchange require intersection controls at the freeway or interstate ramps. Depending on the anticipated traffic demand and the functional types of the intersecting facilities, these could be a simple stop control, signalized, roundabout, DDI, or CFI intersection.

10.3.4 Multimodal Considerations

Accommodations for transit, including buses and rail, as well as bicycles and pedestrian traffic must be considered in the design of a grade separation or interchange. It is best practice to provide bicycle and pedestrian facilities outside the general-purpose lanes of a freeway, such as a shared-use path or the adjacent local street network. It is important to consider safety, such as recovery areas for vehicles, and noise when determining placement of a bicycle or pedestrian facility. Grade separations may be appropriate for bicycle and pedestrian traffic. Although generally discouraged, under Colorado law, when no secondary routes are available, bicyclists and pedestrians may use freeways unless otherwise posted.

Use this link to access Colorado Bicycles and Byways Map: [Colorado Bicycle & Byways Map (coloradodot.info)]

10.4 Access Separations and Control on the Crossroad at Interchanges

Safely manage and minimize weaving by providing appropriate spacing between adjacent intersections and the interchange.

For further information, refer to Chapter 10, Section 4, of the 2018 AASHTO GDHS.

10.5 Safety

- Minimize crossing and turning conflicts.
- Points of access.

For further information, refer to Chapter 10, Section 5, of the 2018 AASHTO GDHS.

10.6 Stage Development

The design of a grade separation or interchange for a project that may be built in stages needs to take into consideration the desired ultimate configuration so that the earlier phases of the project do not preclude future phases.

For further information, refer to Chapter 10, Section 6, of the 2018 AASHTO GDHS.

10.7 Economic Factors

- *Initial Costs.* Most costly type of intersection. Does the need justify the cost?
- *Maintenance Costs.* Increased maintenance and operations costs. Is there adequate equipment and staffing for this or is there a plan to augment the budget for the addition of an interchange?
- *Vehicular Operating Costs.* Different interchange configurations can have a significant cumulative effect in vehicle operating costs. Which alternative minimizes the vehicle operating and overall combined costs of the interchange options.

For further information, refer to Chapter 10, Section 7, of the 2018 AASHTO GDHS.

10.8 Grade Separation Structures

It is recommended to conduct a detailed study for a proposed grade separation to determine which of the intersecting roadways should be the grade-separated structure. Often, the choice is dictated by such features 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.

When initiating the interchange analysis there are a lot of tools to help the designer identify the right interchange solution that fits the context, traffic, and roadway features. Much of this begins with initial traffic analysis tools like FHWA's Cap-X Tool or other similar tools that can evaluate current and future traffic volumes and help to prioritize the best interchange types for the anticipated traffic conditions.

Use this link to access FHWA's Capacity Analysis for Planning of Junctions (CAP-X) Tool User Manual: http://www.cmfclearinghouse.org/collateral/FHWA-SA-18-067%20CAP-X%202018%20Tool%20User%20Guide%20(Final).pdf

Use this link to access FHWA's Capacity Analysis for Planning of Junctions (CAP-X) Tool Software: https://www.fhwa.dot.gov/software/research/operations/cap-x/

As a rule, an interchange design that best fits the existing topography is the most economical to construct and maintain, and this factor is the first consideration in design. Notwithstanding notable exceptions such as the Central 70 project in Denver, another general rule is that the facility with the lesser footprint is typically elevated above or depressed below the facility with the greater footprint. Bear in mind, all other items equal, elevated or depressed freeway area is typically more costly compared to at-grade freeway facilities with a larger footprint/acreage. Every effort should be made to design the grade separation structure so that it fits the environment and context in a pleasing and functional manner without drawing unnecessary or distracting attention.

The type of structure best suited to a grade separation is one that maintains a constant roadway width and typical section. The clear width on bridges should be as wide as the approach pavement, including shoulders, so that a driver has a secure feeling. When the full approach roadway is continued across the structure, the protective railing or parapet rail on both sides of the roadway should align with the guardrail on the approach roadway to achieve a uniform appearance.

Minimum lateral clearances at underpasses and retaining walls should include any provisions for the dynamic lateral deflection that the guardrail may require. Refer to Section 8.4 of this Guide for information on vertical clearances.

For more information on grade separation structures, refer to Chapter 10 of the 2018 AASHTO GDHS.

10.9 Interchanges

10.9.1 General Considerations

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. An interchange should have one point of exit located in advance of the crossroad wherever practical. Along with the interchange design, it is desirable to reconfigure portions of the local street system to achieve an effective overall plan of traffic service and community development.

The interchange 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.

There are several basic interchange forms or ramp geometric patterns for turning movements at an interchange. 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, signing, context classification, and the designer's judgment.

From the standpoint of safety and preventing wrong-way movements, freeway interchanges with non-access-controlled highways should have ramps to serve all basic directional movements. A specific freeway-to-freeway movement may be omitted if the turning traffic volume is minor and can be accommodated via route signing to 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 semi directional interchanges.
- Offset interchanges.

- Combination interchanges.

CDOT and FHWA discourage the use of partial interchanges, and these should be avoided. Refer to Chapter 10 of the 2018 AASHTO GDHS.

The CSS process can help a designer determine the best interchange type for the context it is within while including stakeholders in the discussion. Understanding the stakeholders needs will lead to the design that will best accommodate the highest demands for operations and safety.

10.9.2 Three-Leg Designs

Refer to Chapter 10, Section 10.9.2, of the 2018 AASHTO GDHS.

10.9.3 Four-Leg Designs

Refer to Chapter 10, Section 10.9.3, of the 2018 AASHTO GDHS.

10.9.4 Other Interchange Configurations

Refer to Chapter 10, Section 10.9.4, of the 2018 AASHTO GDHS.

10.10 General Design Considerations for Interchanges

Except for crash data, the same basic data for intersection design considerations covered in Chapter 8 of this Guide is required for interchange design. This includes:

- Design speed.
- Design traffic volumes.
- Level of service.
- Pavement and shoulders.
- Curbs.
- Superelevation.
- Grades.
- Structures.
- Horizontal and vertical clearance sight distance.

Data that should be obtained prior to interchange design includes 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 (context sensitive solutions). Specifically, this includes:

- The location and standards (types) of existing and proposed local streets and highway development, including types of traffic (access) control.

- Existing and potential traffic circulation and multimodal connectivity 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 current and future (typically 20-year) annual average daily traffic, design hourly volumes, level of service, 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 and intersections.
- The location of major utilities and multimodal facilities (e.g., railroads, transit, airports).

10.10.1 Determination of Interchange Configuration

Interchanges may be implemented on all functional roadway classifications, as discussed in Section 10.2 of this chapter.

In rural areas, interchange-type selection is determined based on service demand. The common rural interchange type in use in Colorado is the diamond interchange. Interchange-type determination in an urban environment requires considerable analysis of regional conditions so that the most practical interchange configurations can be developed.

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 are not created.

Cloverleaf interchanges are not commonly used anymore. Proper spacing between the entry and exit points on loops are hard to achieve without creating a large interchange footprint. Also, the size of the loops needs to be large to accommodate faster vehicle travels speeds to prevent roadway departure crashes. There are many alternative interchange designs that do not require these large footprints and operate effectively to manage traffic. A simple diamond interchange is 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. It is becoming more common to see roundabouts as an intersection type with diamond interchanges in rural areas.

Another intersection alternative for diamond interchanges is the displaced left-turn also known as a continuous flow intersection (CFI) configuration. The benefit of a CFI intersection is the ability to combine movements that do not conflict with one another to reduce the number of phases required to move all the traffic. This benefits operations for the overall interchange.

A DDI is a variation of a conventional diamond interchange. 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, DDIs typically have higher left-turn volume capacity and improved operations compared to conventional signalized diamond interchanges because of shorter cycle lengths, improved safety through the elimination of turning conflicts, reduced time lost per cycle phase, reduced stops and delay, and shorter queue lengths.

A SPUI is an interchange with 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 from a four-phase to a three-phase operation. Due to their large footprint, a SPUI on surface grade is preferred to a SPUI on an elevated structure.

A partial cloverleaf interchange 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 without any appreciable effect to or from other interchanges within the system.

The final decision on the interchange configuration 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.

10.10.2 Multimodal Considerations

Because interchanges are often receiving or dispersing vehicles from local roadways, arterials and collectors to the freeway system, it is necessary to consider how transit, bicycle, and pedestrian modes using the local roadways will navigate the interchange. The CSS and PBPD processes can determine modal priorities that lead to the best design.

Refer to Chapter 10 of the 2018 AASHTO GDHS.

10.10.3 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 grade separation 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. Regarding order of precedence in geometric design, it is most common to allow major highway or mainline geometrics to control interchange geometrics over the secondary roadway. Interchange designers are sometimes tempted to not follow this protocol as they struggle to establish the interchange geometrics. For example, drastically raising the mainline elevation may make flyover geometrics easier to design but this will likely be a more costly proposition for the DOT overall.

The general controls for horizontal and vertical alignment and their combination, as stated in Chapter 3 of this Guide, 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 depends 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 in Chapter 10 of the 2018 AASHTO GDHS. This table provides 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 minimum 20- to 22-foot difference in elevation is needed at a grade separation of two highways for essential vertical clearance and structure depth. This is also a good rule of thumb when considering the overall height of a multilevel interchange. For example, one could estimate that the upper-most driving surface of a four-level interchange is approximately between 80 to 88 feet high. The same 20- to 22-foot dimension generally applies to a single-level highway undercrossing a railroad; however, about 28 feet is needed for a highway overcrossing a railroad. However, input from bridge designers is required to determine relevant final design considerations.

10.10.3.1 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 8 of this Guide.

10.10.4 Interchange Spacing

In general, the desired 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.

10.10.5 Uniformity of Interchange Patterns

To the extent practical, all interchanges along the freeway should be reasonably uniform in geometric layout and general appearance. Except in special cases, all entrance and exit ramps should be on the right. Left entrances are undesirable because of difficulties related to merging with high-speed through traffic.

10.10.6 Route Continuity

Refer to Chapter 10 of the 2018 AASHTO GDHS.

10.10.7 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. 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 along a corridor.

Taking the extra step to evaluate upstream and downstream traffic operations and to model those operations with modeling software can avoid the potential of lane imbalance at interchange ramps and connections. This will prevent secondary crashes upstream by preventing the bottleneck condition if addressed properly during the early design analysis stage of the interchange.

10.10.8 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, or 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 exiting traffic. Where auxiliary lanes are provided along freeway mainlines, the adjacent shoulder width should be 8 to 12 feet in width, with a minimum 6-foot-wide shoulder in width-restricted areas.

Proper development of auxiliary lanes on the secondary road can greatly reduce weaving and queueing conflicts improving the operations on the secondary road around the interchange.

10.10.9 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. It is important to remember that the lane reduction should not be made so far downstream that drivers become accustomed to a certain 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 drivers' unfamiliarity with left-side merges.

10.10.10 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.

A simple weaving section has an entrance at the upstream end and an exit at the downstream end. A multiple weaving section has more than one point of entry followed by one or more points of exit. For the various types of weaving situations, refer to Figure 10-2A and Figure 10-2B.

Figure 10-2A Types of Weaving Sections

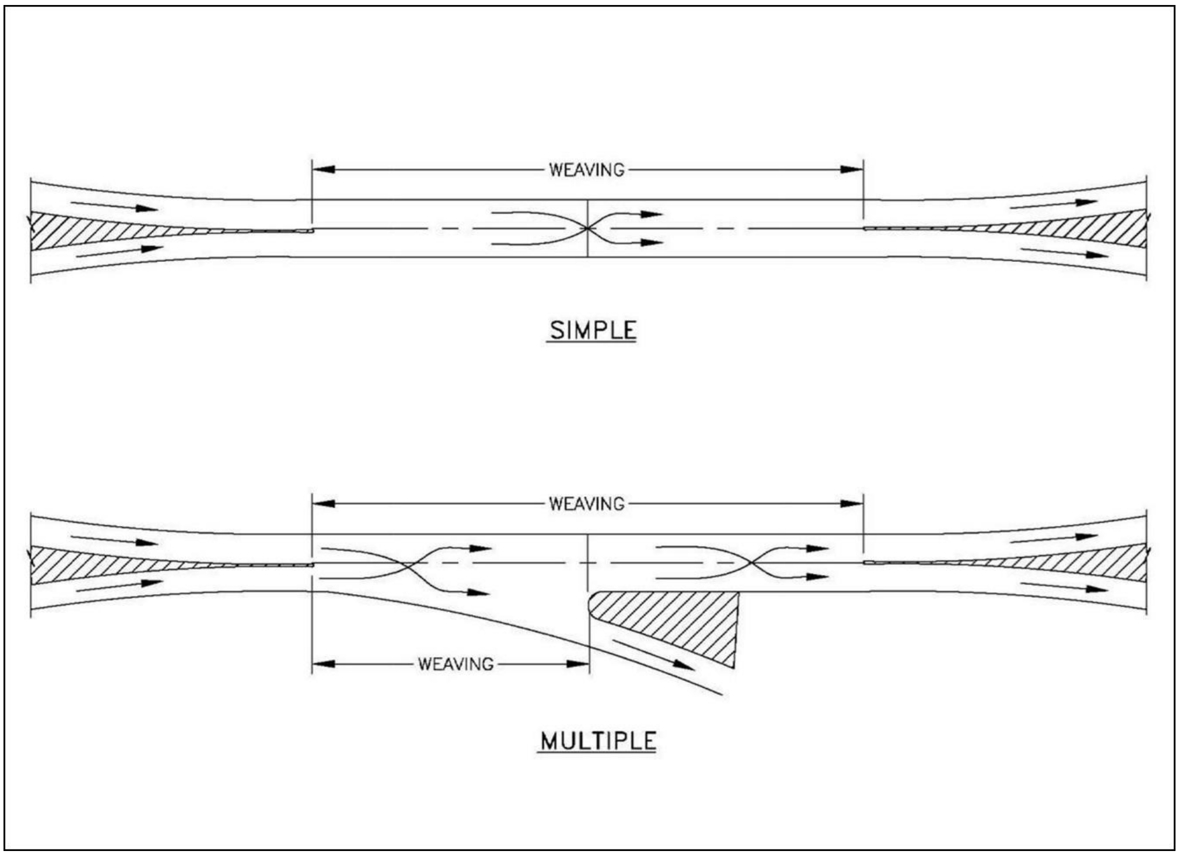
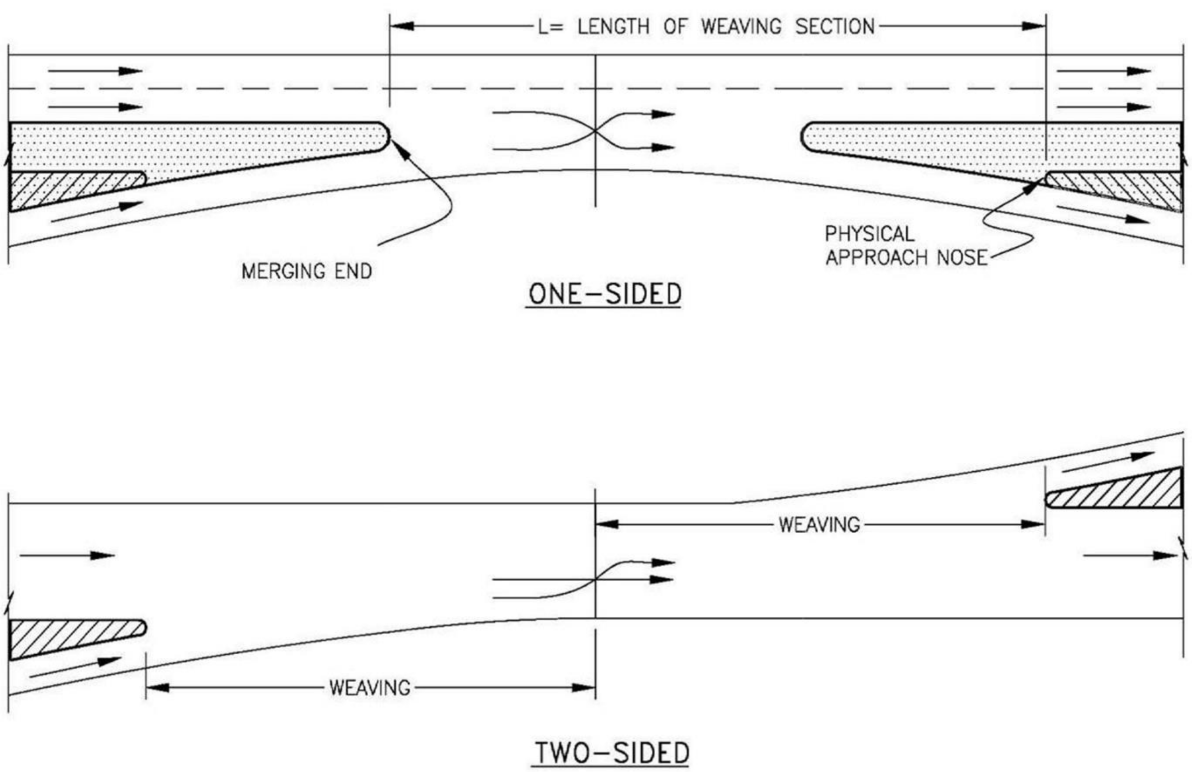


Figure 10-2B Types of Weaving Sections



Adequate length and width are required through the weaving section, along with lane balance, to maintain adequate capacity of weaving sections. Refer to Chapter 2 of the 2018 AASHTO GDHS for how to determine weaving lengths and widths.

Figure 10-2C and Figure 10-2D are 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* (TRB, 2016). Weaving sections in urban areas should be designed for LOS C or D where possible. Weaving sections in rural areas should be designed for LOS B or C. Volume in equivalent passenger cars per hour (PCPH) is adjusted for freeway grade and truck volumes.

The CDOT Region Traffic and Safety Engineer should be consulted for difficult weaving analysis problems.

Figure 10-2C Lane Configuration of Weaving Sections

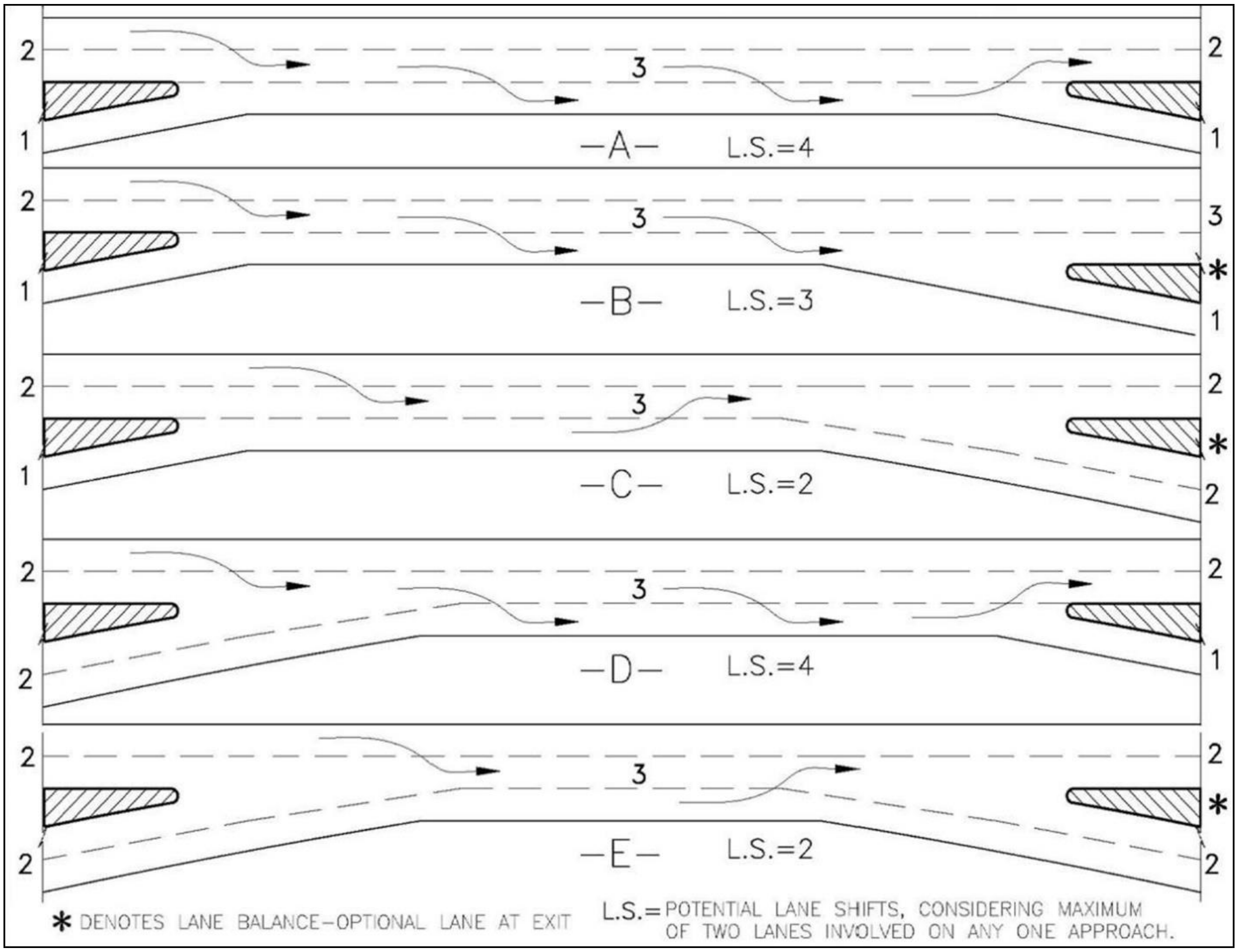
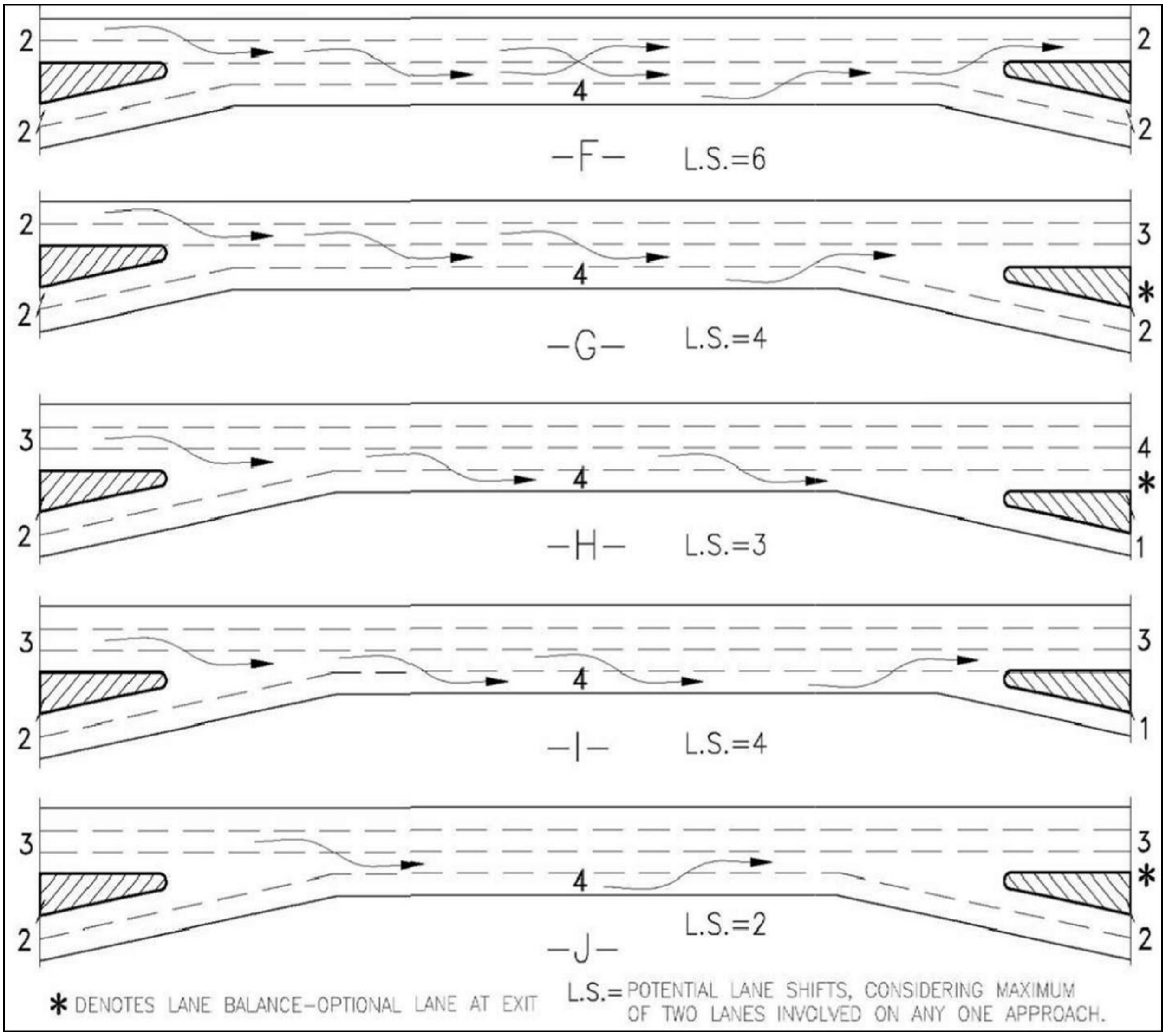


Figure 10-2D Lane Configuration of Weaving Sections



10.10.11 Collector-Distributor Roads

Collector-distributor roads between two interchanges and continuous collector-distributor roads are discussed in Chapters 8 and 10 of the 2018 AASHTO GDHS.

10.10.12 Two-Exit Versus Single-Exit Interchange Design

In general, interchanges that are designed with single exits are preferred over 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.

When designing a two-lane exit, it is advantageous to develop a single drop lane (right most lane) and a "choice lane" in which the motorist in the second (left hand lane) can either exit

to the interchange or continue past the interchange along the roadway. This can help to avoid sudden lane changes at the last minute when a motorist is deciding to exit or not and will reduce the potential for high-speed side-swipe crashes due to sudden lane changes.

10.10.13 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 crash. 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.

Refer to Chapter 10 of the 2018 AASHTO GDHS for more information.

10.11 Ramps

10.11.1 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, refer to Chapter 10 of the 2018 AASHTO GDHS.

10.11.2 General Ramp Design Considerations

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 at this end of this section. 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.
- *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 8 of this Guide. In urban areas, where the land adjacent to the interchange is developed commercially, provisions for pedestrian movements through the interchange area should be considered.
- *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-foot radius). If less restrictive conditions exist, the loop design speed and the radius may be increased.

If lower-range design speeds are used for ramps, consideration should be made for additional acceleration/deceleration length and warning signs.

Refer to Chapter 10 of the 2018 AASHTO GDHS on Curvature.

10.11.3 Bicyclist and Pedestrian Integration

Integration of pedestrians and bicyclists needs strong consideration due to the inherent nature of the high vehicle speeds on facilities with interchanges and the driver expectations that they will not encounter pedestrians or cyclists. To provide the safest environment for the pedestrian and bicyclist, it is advisable to include separated shared-use paths that run parallel to the freeway; and to provide traffic signals with pedestrian/bicycle phases in the signal timing at ramp terminal intersections with high traffic volumes so pedestrians and bicyclists can safely cross the ramp terminals.

10.11.4 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 6-1 should 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 8 of this Guide or the 2018 AASHTO GDHS.

10.11.5 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%.
- Upgrades on ramps having a design speed of 40 mph should be limited to 4 to 6%
- Upgrades on ramps having design speeds of 25 to 30 mph should be limited to 5 to 7%.
- Upgrades on ramps having a design speed of 15 to 25 mph should be limited to 6 to 8%.
- Downgrades preferably should be limited to 3 to 4% on ramps with sharp horizontal curvature and significant heavy truck or bus traffic. Short upgrades of as much as 5% 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 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, refer to Chapter 9, Figure 9-44 through 9-47 of the 2018 AASHTO GDHS.

In addition, an aesthetics analysis of each alternate preliminary plan should be conducted. A ramp design that meets all design requirements may have objectionable features that can be eliminated with small changes. Examples of objectionable features are:

- Humps or rolls in a ramp profile.
- Short reverse curvature in ramp alignment.
- Abrupt grade changes when ramp termini profile meets crossroad cross slope.

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 because it is impossible to obtain proper superelevation of the curves without an intervening length of tangent for superelevation transitions between the reversing curves.

10.11.6 Ramp Horizontal Geometrics and Superelevation

Ramp design should be looked at as a three-dimensional system to analyze the facility to function as anticipated. For example, the ramp gores and sight distance need to be evaluated from a safety aspect and may require the designer to provide sight distances greater than the minimum stopping sight distance and using above minimum design criteria for other geometric elements. The vertical and horizontal coordination is particularly critical when horizontal curves occur at the end of a downgrade and at the top of a vertical curve. The maximum speed differential between adjacent alignment elements should not exceed 10 miles/hour. These conditions are typical of interchange ramp design. Each ramp terminal is considered its own intersection design for interchanges unless signalized and signed with no right turn on red.

10.11.7 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 (CDOT, 2019). Where drainage impacts to adjacent property or the frequency of slow-moving vehicles are important considerations, the superelevation rates and corresponding radii in Figure 6-16 of the 2018 AASHTO GDHS can be used.

- The superelevation development starts or ends along the auxiliary lane of the ramp terminal. Alternate profile lines for both edges should be studied so that all profiles match the control points and there are no unsightly bumps and dips. Spline profiles are very useful in developing smooth lanes and shoulder edges.
- Another important control in developing superelevation along the ramp terminal is 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 10-1.
- 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.

Table 10-1 Maximum Algebraic Difference in Pavement Cross Slope at Turning Roadway Terminals

Design Speed of Exit or Entrance Curve (mph)	Maximum Algebraic Difference in Cross Slope at Crossover Line (%)
20 and under	5.0 to 8.0
25 and 30	5.0 to 6.0
35 and over	4.0 to 5.0

10.12 Interchange Design Criteria

10.12.1 General

The following design criteria pertain to design elements of all grade separations and 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 described in Section 10.11 of this chapter.

Grade separations and interchanges have a combination of channelization elements. Design criteria pertaining to intersections at grade are described in Chapter 8 of this Guide; design criteria unique to interchanges are included in this section.

10.12.2 Multimodal Considerations

Chapter 13 of this Guide provides design criteria for bicycle and pedestrian facilities.

10.12.3 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 2018 AASHTO GDHS, 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 is further limited because of 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 are allowed on overpass structures for the lateral offset from the lane line to the bridge rail. Exceptions can be made for future lanes or for construction phasing requirements when replacing existing structures. Refer to Section 8.3 for additional information about lateral clearances on structures.

10.12.4 Sight Distance to Exit Nose

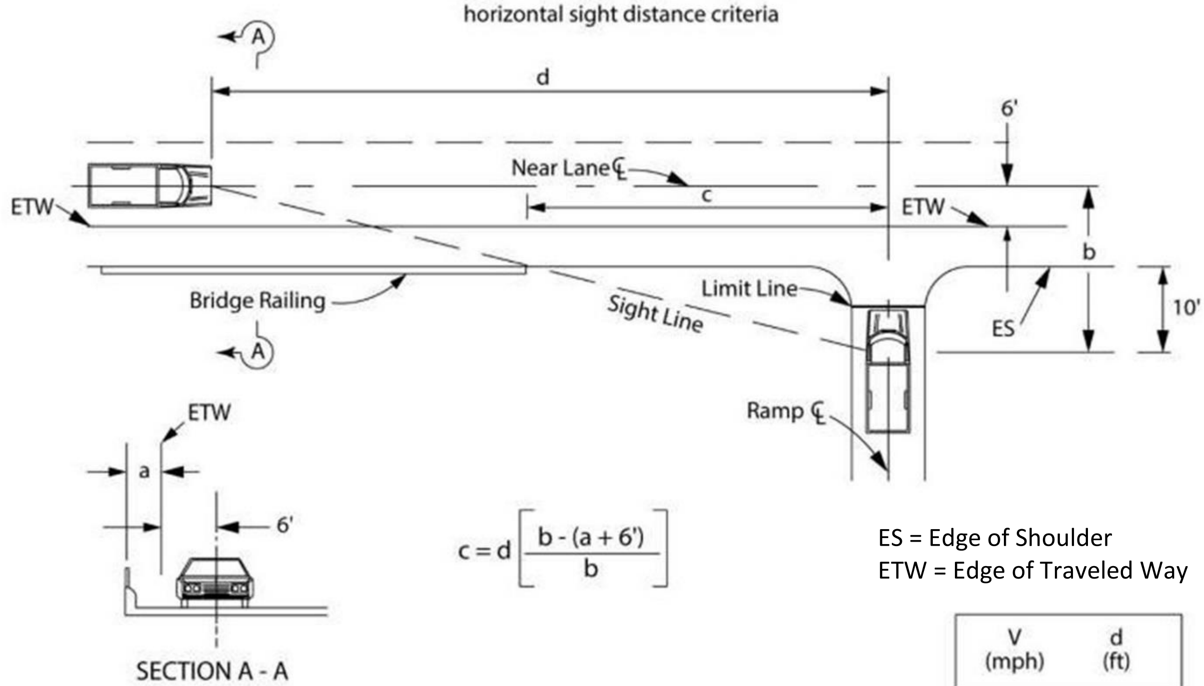
A clear view of the entire exit ramp terminal is desired, including the exit nose and a section of the ramp pavement beyond the gore. The sight distance on a freeway preceding the approach nose of an exit ramp should exceed the minimum for the through traffic design speed, desirably by 25% or more. In addition, the minimum sight triangle shown in Figure 10-3 and Figure 10-4 should be provided between vehicles approaching the ramp intersections. For considerations of longer sight distances, refer to Chapter 3 of this Guide.

Decision sight distance given in Chapter 6 of this Guide 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. Refer to Chapter 8 of this Guide.

Figure 10-3 Location of Ramp Intersections on the Cross Road

Location of Ramp Intersections on the Crossroads

Unsignalized and based on 7.5 second horizontal sight distance criteria



V (mph)	d (ft)
25	275
30	330
35	385
40	440
45	495
50	550
55	605
60	660
65	715
70	770

a = Distance from edge of traveled way to bridge railing.

b = Distance from center of near lane to eye of ramp vehicle driver. Ramp driver's eye is assumed to be located 10′ from the edge of shoulder, but not less than 15′ from the ETW (therefore, b = 6′ + shoulder width + 10′). See Index 405.1.

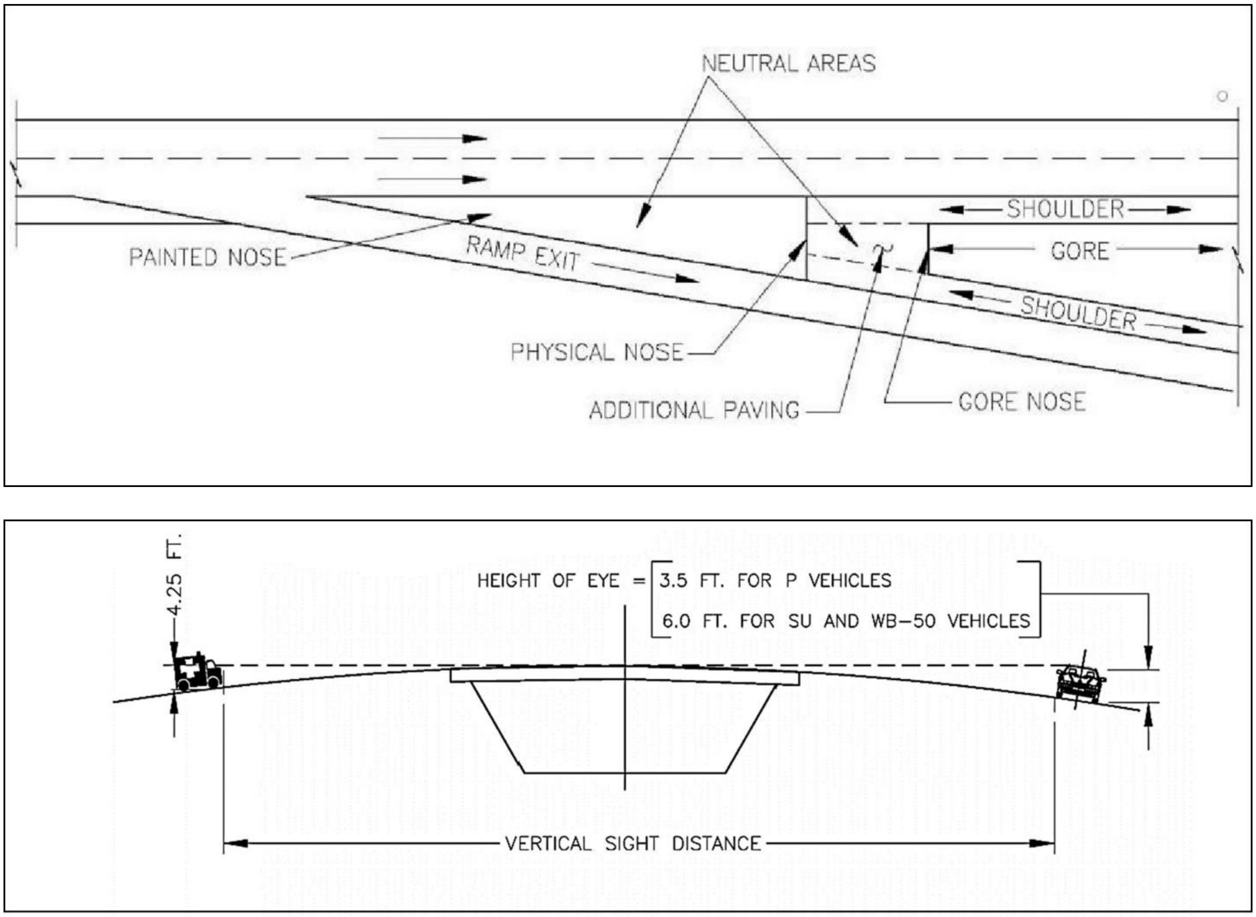
c = Ramp set back from end of bridge railling.

d = Corner Sight distance along highway from intersection. (See Table above) Sight distance is measured from a 3½′ eye height on the ramp to a 4¼′ object height on the crossroad.

V = Anticipated prevailing speed on crossroad.

ETW = Edge of Traveled Way; ES = Edge of Shoulder.

Figure 10-4 Measurement of Sight Distance at Ramp Terminals with Stop Control

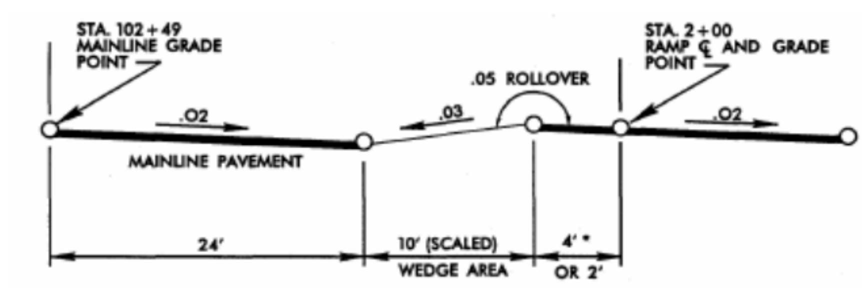


10.12.5 Gores

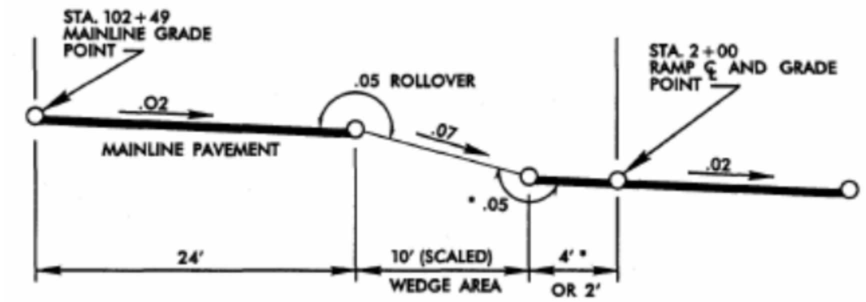
The term "gore" indicates an area downstream from the shoulder intersection points, as illustrated in Figure 10-5.

Figure 10-5 Typical Gore Area Controls

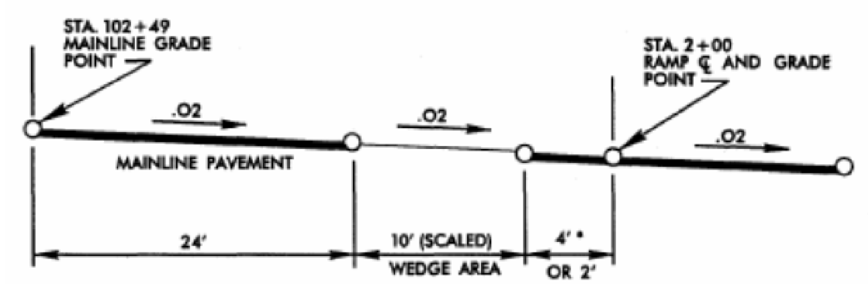
Maximum Gore Controls



Minimum Gore Controls



Desirable Gore Controls



It is the decision point area that must be clearly seen and understood by approaching drivers. Further, the separating ramp roadway must be clearly evident and 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 general appearance.

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.

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.

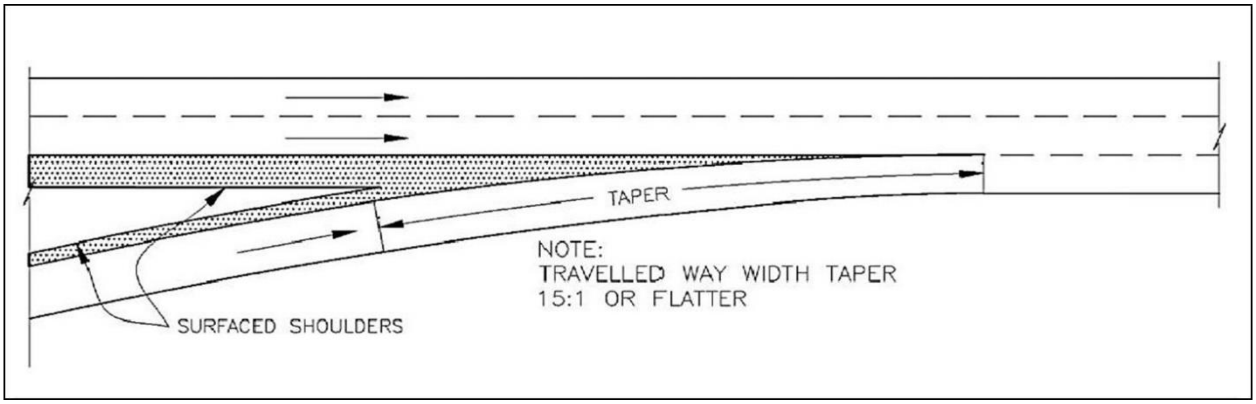
It is imperative that gore areas and the areas beyond provide clear recovery area for out-of-control-vehicles or for drivers who decide at the last second not to exit. Additional paving should be placed in the neutral area between the physical nose and the gore nose to allow drivers to recover after starting their exit maneuver.

The gore area and the unpaved area beyond should be kept as free of obstructions as possible to create 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 are not 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.

The term "gore" can refer to the area between a through roadway and an exit ramp or between a through roadway and a 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 limited to the sum of the shoulder widths on the ramp and freeway plus a narrow physical nose 4 to 8 feet wide.

Figure 10-6 diagrammatically details a typical gore design for an entrance ramp.

Figure 10-6 Traveled Way Narrowing on Entrance Ramps



10.12.6 Ramp Pavement Widths

10.12.6.1 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. Refer to Chapter 3 of this Guide 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 8-1 (Design Widths of Pavements for Turning Movements) of this Guide.

10.12.6.2 Shoulders and Lateral Clearances

Design values for shoulders and lateral clearances on the ramps are as follows:

- Paved shoulders on ramps 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 2018 AASHTO GDHS for Case II and Case III should be modified when there are paved shoulders 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 of 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. At a minimum, structural supports should be at least 4 feet beyond the edge of paved shoulder. The AASHTO Roadside Design Guide (AASHTO, 2011) 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 are desirable.

10.12.7 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 at-grade ramp terminals are discussed in Chapter 8 of this Guide; Free-flow ramp terminals are discussed in the following sections.

Terminals are further defined 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.

10.12.7.1 Right-Hand Entrances and Exits

Freeway entrances and exits typically connect to the right of through traffic. Right-hand entrances and exits operate fairly well and do not violate the concept of driver expectancy.

10.12.7.2 Left-Hand Entrances and Exits

Because left-hand entrances and exits are contrary to driver expectancy, it is recommended to avoid their use in an interchange and on high-speed, free-flow ramp terminals. Special attention should be given to the weaving movements from adjacent right-hand entrances or exits, appropriate advanced signing, and adequate decision sight distance that can alert the driver to an unusual situation.

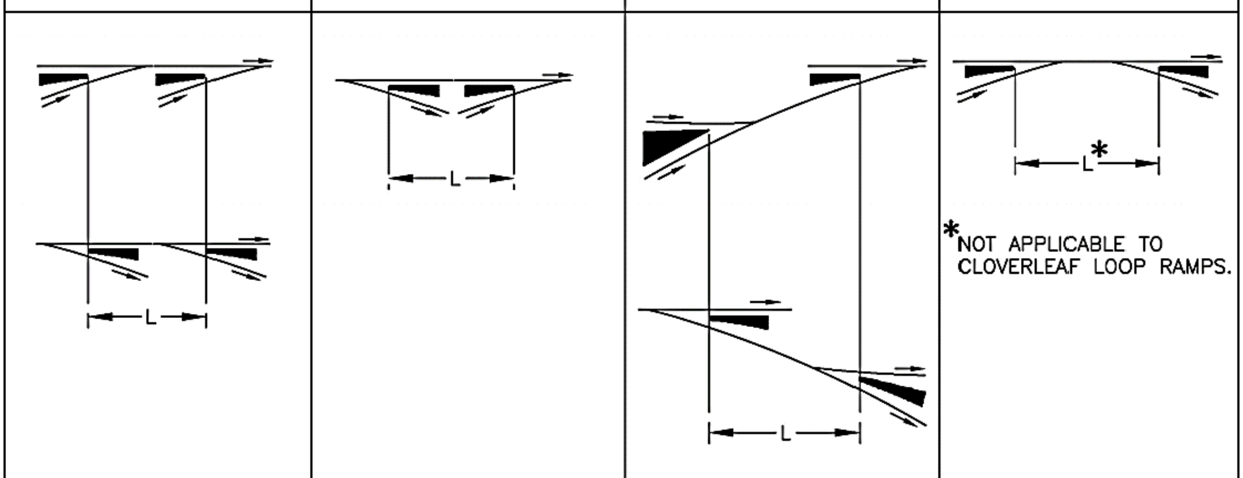
At major forks and branch connections, the less significant roadway should exit and enter on the right. Refer to the discussion on route continuity in Chapter 10 of the 2018 AASHTO GDHS.

Left-side terminals break up the uniformity of interchange patterns and, in general, result in driver hesitancy. Left-hand entrances and exits in an interchange may be considered in unusual circumstances or for collector-distributor roads.

10.12.7.3 Terminal Location and Sight Distance

Freeway ramp 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 should conform to Figure 10-7 wherever practical.

Figure 10-7 Recommended Minimum Ramp Terminal Spacing



EN–EN OR EX–EX		EX–EN		TURNING ROADWAYS		EN–EX (WEAVING)			
FULL FREEWAY	CDR OR FDR	FULL FREEWAY	CDR OR FDR	SYSTEM INTERCHANGE	SERVICE INTERCHANGE	SYSTEM TO SERVICE INTERCHANGE		SERVICE TO SERVICE INTERCHANGE	
						FULL FREEWAY	CDR OR FDR	FULL FREEWAY	CDR OR FDR
MINIMUM LENGTHS MEASURED BETWEEN SUCCESSIVE RAMP TERMINALS (FEET)									
1000	800	500	400	800	600	2000	1600	1600	1000

EN = Entrance, CDR = Collector/Distributor Road, FDR = Freeway Distributor

10.13 Speed-Change Lanes

There are two general types of speed-change lanes: the taper and the parallel. The taper type has 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. Minimum lengths are provided in Table 10-2 for the ramp types shown in Figure 10-8 and in Figure Table 10-3 for the ramp types shown in Figure 10-9. Table 10-4 identifies corrections to be applied when the speed-change lane grades are 3% or steeper.

Table 10-2 Minimum Deceleration Lengths for Exit Terminals with Flat Grades of 2% or Less

Highway Design Speed, mph (V)	Speed Reached, mph (V_A)	L = Deceleration Length, (ft) for Design Speed of Exit Curve, mph (V')								
		Stop Condition	15	20	25	30	35	40	45	50
		For Average Running Speed on Exit Curve, mph (V'_A)								
		0	14	18	22	26	30	36	40	44
30	28	235	200	170	140	---	---	---	---	---
35	32	280	250	210	185	150	---	---	---	---
40	36	320	295	265	235	185	155	---	---	---
45	40	385	350	325	295	250	220	---	---	---
50	44	435	405	385	355	315	285	225	175	---
55	48	480	455	440	410	380	350	285	235	---
60	52	530	500	480	460	430	405	350	300	240
65	55	570	540	520	500	470	440	390	340	280
70	58	615	590	570	550	520	490	440	390	340
75	61	660	635	620	600	575	535	490	440	390

Figure 10-8 Ramp Types for Table 10-2 (Exit Terminals)

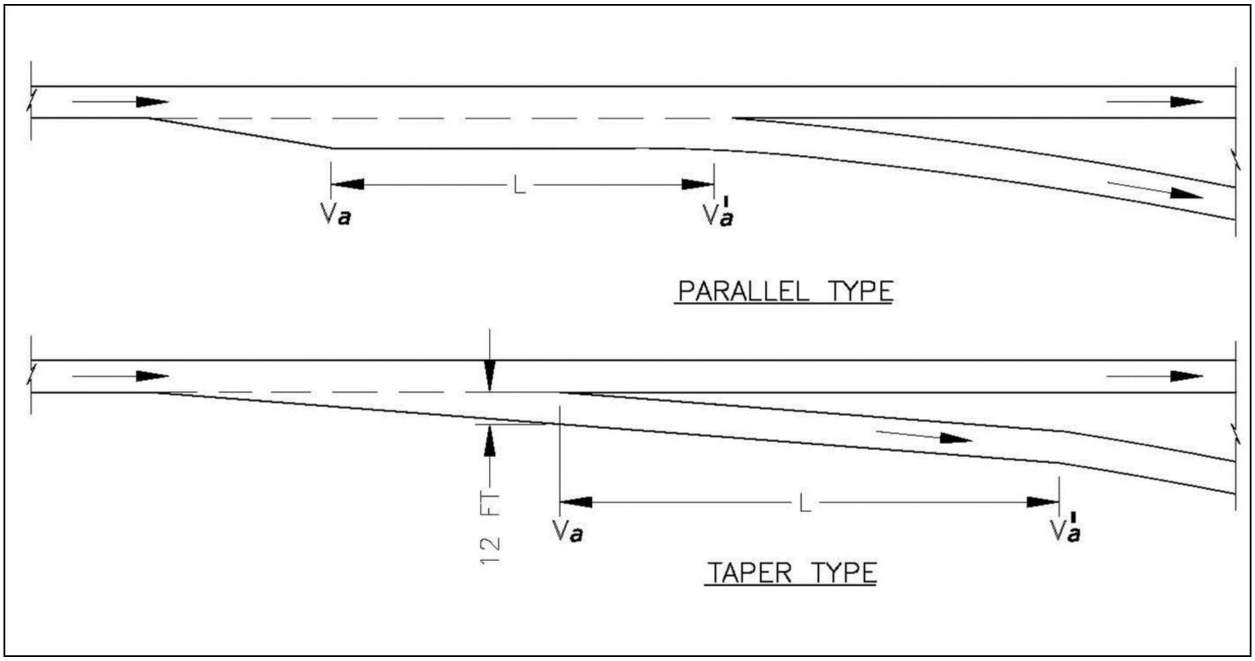


Table 10-3 Minimum Acceleration Lengths for Entrance Terminals with Flat Grades of 2% or Less

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.

Figure 10-9 Ramp Types for Table 10-3 (Entrance Terminals)

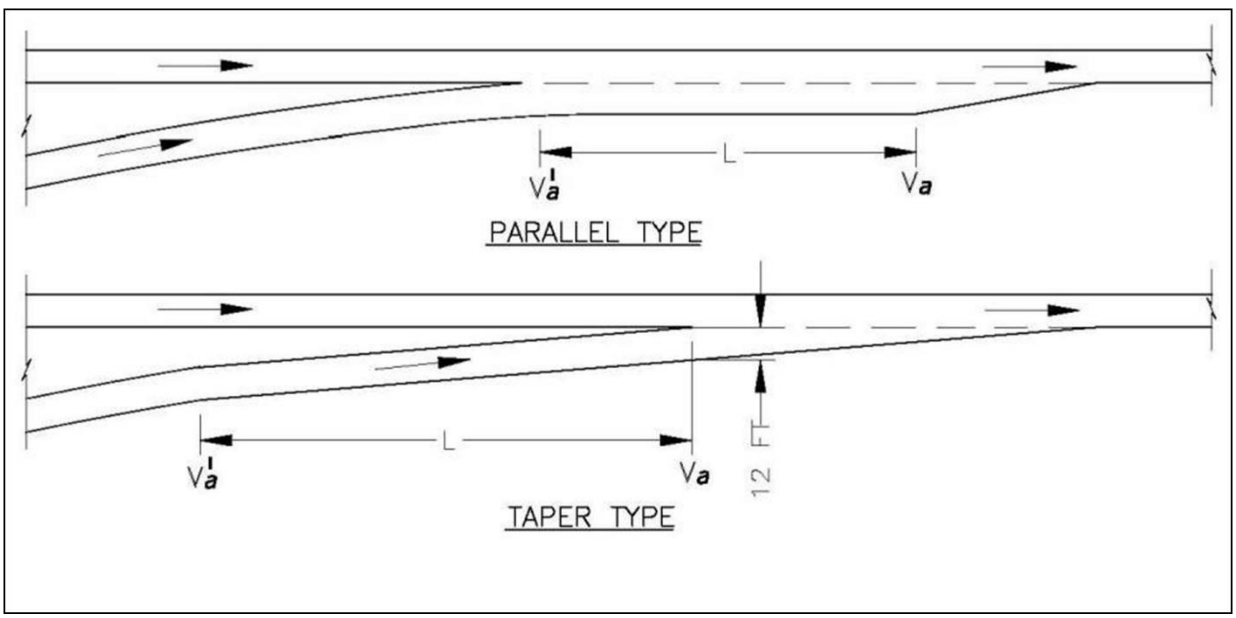


Table 10-4 Speed-Change Lane Adjustment Factors as a Function of Grade (All Terminal Types)

DESIGN SPEED OF HIGHWAY (mph)	**Ratio of Length on Grade to Length on Level for Design Speed of Turning Curve (mph)[a]**				
All speeds	3 to 4% UPGRADE 0.9				3 to 4% DOWNGRADE 1.2
All speeds	5 to 6% UPGRADE 0.8				5 to 6% DOWNGRADE 1.35
DESIGN SPEED OF HIGHWAY (mph)	**ACCELERATION LANES Ratio of Length on Grade to Length of Level for Design Speed of Turning Curve (mph)***				
	DESIGN SPEED OF TURNING ROADWAY CURVE, mph				
	20	**30**	**40**	**50**	**All speeds**
	3 to 4% UPGRADE				**3 to 4% DOWNGRADE**
40	1.3	1.3	---	---	0.7
45	1.3	1.35	---	---	0.675
50	1.3	1.4	1.4	---	0.65
55	1.35	1.45	1.45	---	0.625
60	1.4	1.5	1.5	1.6	0.6
65	1.45	1.55	1.6	1.7	0.6
70	1.5	1.6	1.7	1.8	0.6
	5 to 6% UPGRADE				**5 to 6% DOWNGRADE**
40	1.5	1.5	---	---	0.6
45	1.5	1.6	---	---	0.575
50	1.5	1.7	1.9	---	0.55
55	1.6	1.8	2.05	---	0.525
60	1.7	1.9	2.2	2.5	0.5
65	1.85	2.05	2.4	2.75	0.5
70	2.0	2.2	2.6	3.0	0.5

* Ratio from this table multiplied by length in Table 10-3 or 10-4 gives length of speed-change lane on grade.

10.13.1 Single-Lane Free-Flow Terminals, Entrances

10.13.1.1 Entrance Ramp Terminals

Design of entrance ramp terminals should conform to the standard designs illustrated in Figure 10-10A, Figure 10-10B, Figure 10-11, and Figure 10-12. Single-lane ramps should be designed for a one-lane, passing-permitted operation. It is up to the designer, with the approval of the project manager, to determine the type of ramp terminal, taper type or parallel type, at each location, although there should be an effort to obtain consistency in a corridor.

Figure 10-10A Typical Single-Lane Entrance Ramps (Tapered)

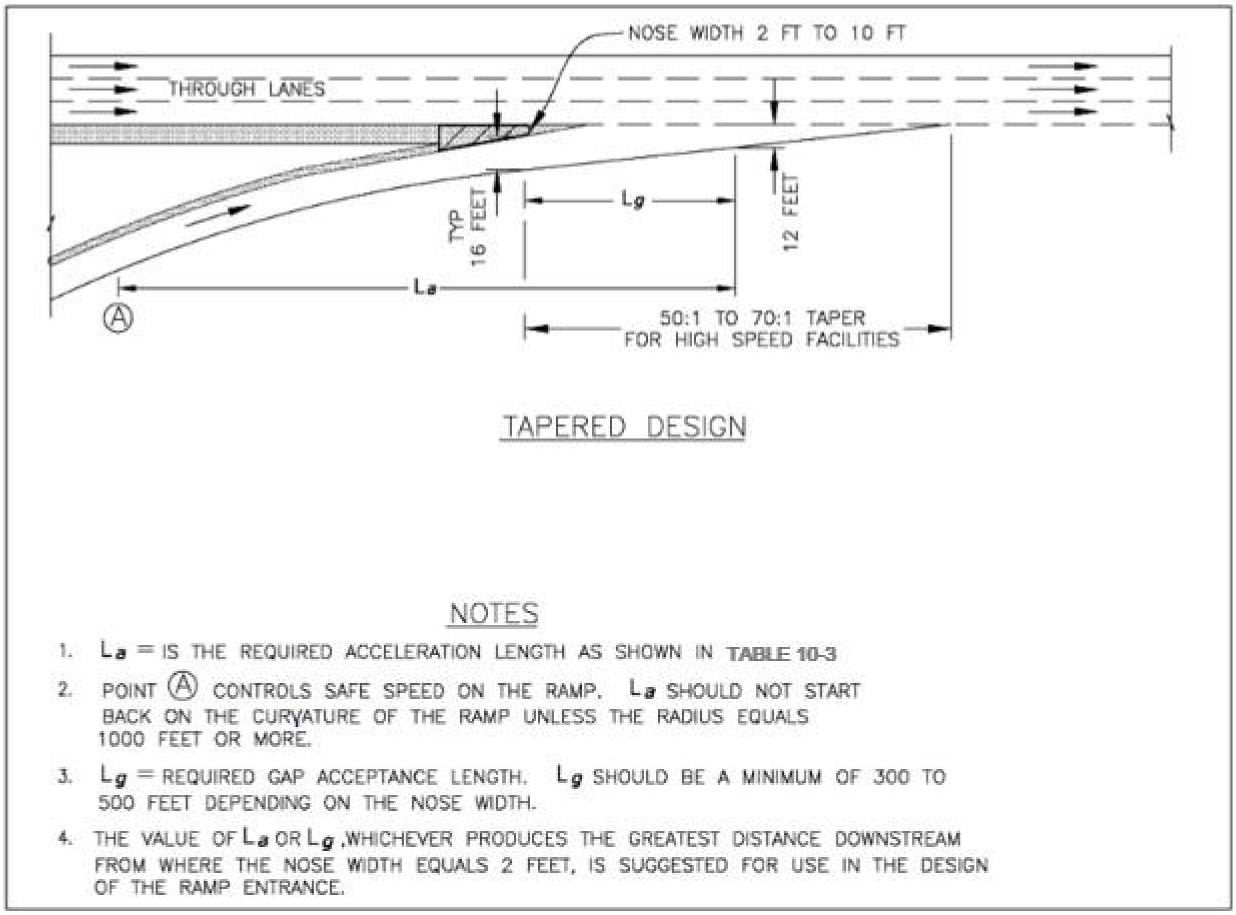
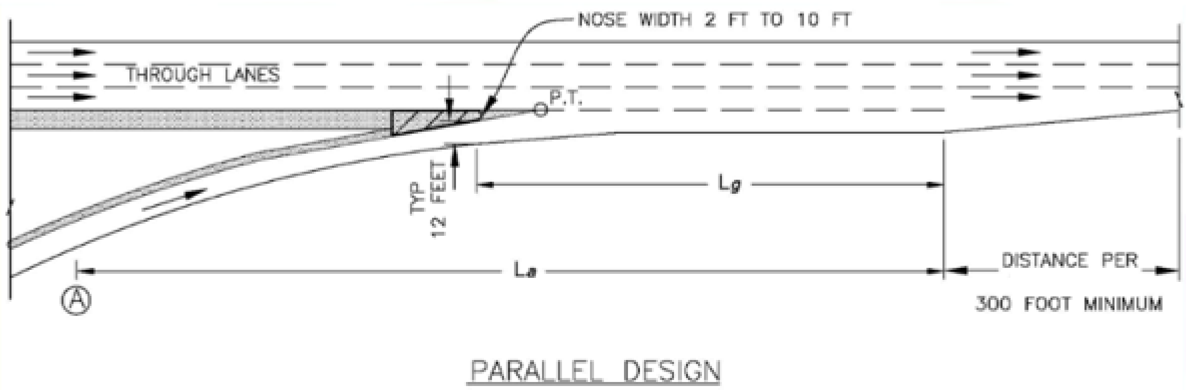


Figure 10-10B Typical Single-Lane Entrance Ramps (Parallel) (Preferred)



NOTES

1. La = IS THE REQUIRED ACCELERATION LENGTH AS SHOWN IN TABLE 10-3
2. POINT Ⓐ CONTROLS SAFE SPEED ON THE RAMP. La SHOULD NOT START BACK ON THE CURVATURE OF THE RAMP UNLESS THE RADIUS EQUALS 1000 FEET OR MORE.
3. Lg = REQUIRED GAP ACCEPTANCE LENGTH. Lg SHOULD BE A MINIMUM OF 300 TO 500 FEET DEPENDING ON THE NOSE WIDTH.
4. THE VALUE OF La OR Lg ,WHICHEVER PRODUCES THE GREATEST DISTANCE DOWNSTREAM FROM WHERE THE NOSE WIDTH EQUALS 2 FEET, IS SUGGESTED FOR USE IN THE DESIGN OF THE RAMP ENTRANCE.

Figure 10-11 Freeway Entrance Terminal (Taper Type)

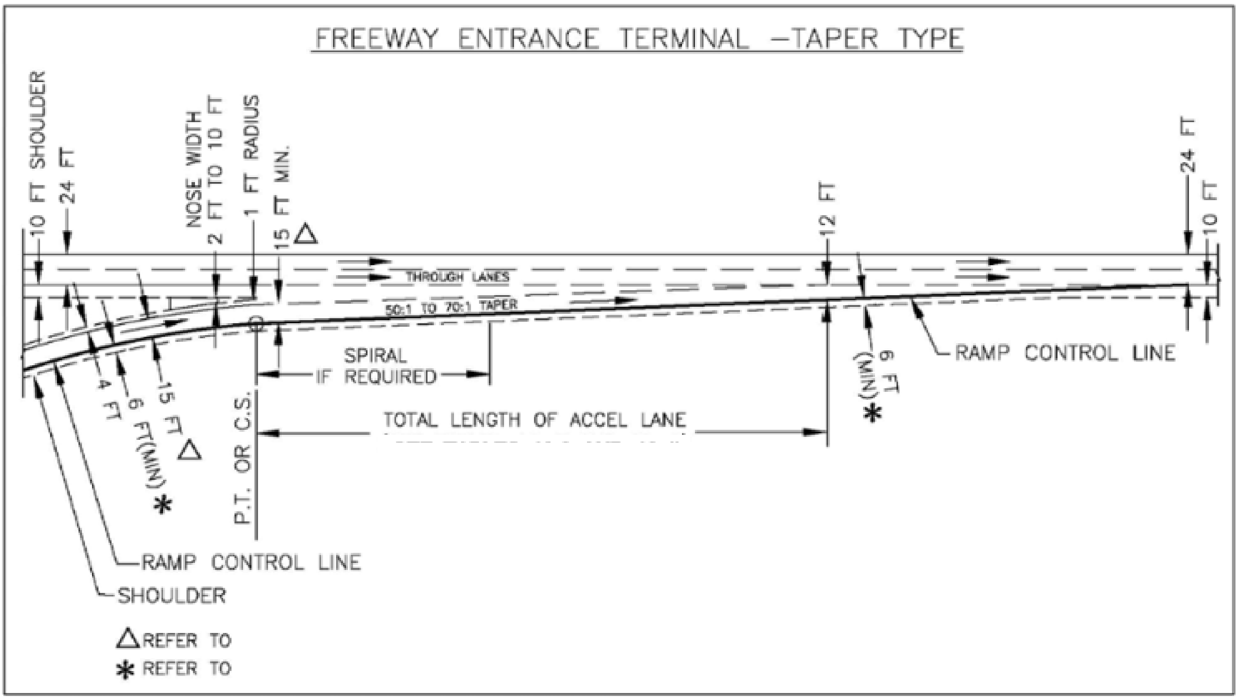
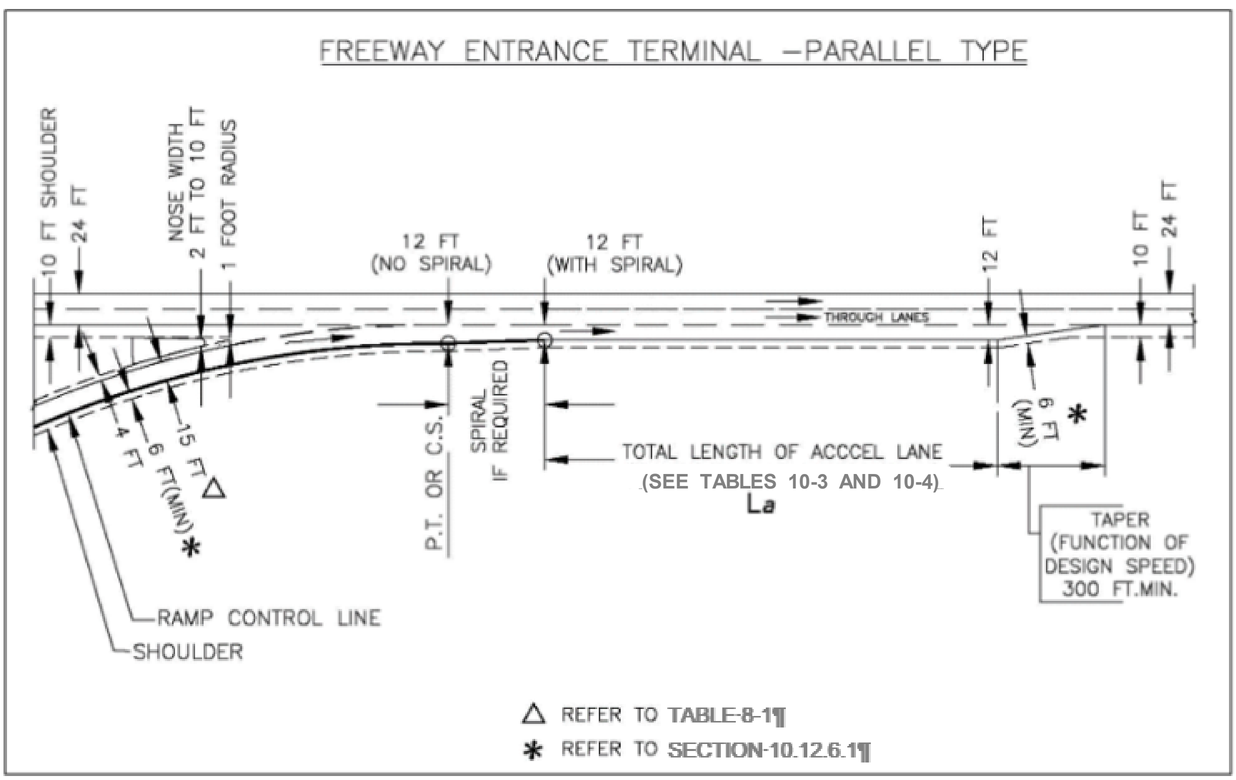


Figure 10-12 Freeway Entrance Terminal (Parallel Type) (Preferred)



10.13.2 Single-Lane Free-Flow Terminals, Exits

10.13.2.1 Exit Ramp Terminals

Design of exit ramp terminals should conform to the standard designs illustrated in Figure 10-13 and Figure 10-14. Single-lane ramps should be designed for a one-lane, passing-permitted operation. Table 10-5 details the minimum length of taper beyond an offset nose.

Figure 10-13 Freeway Exit Terminal (Taper Type)

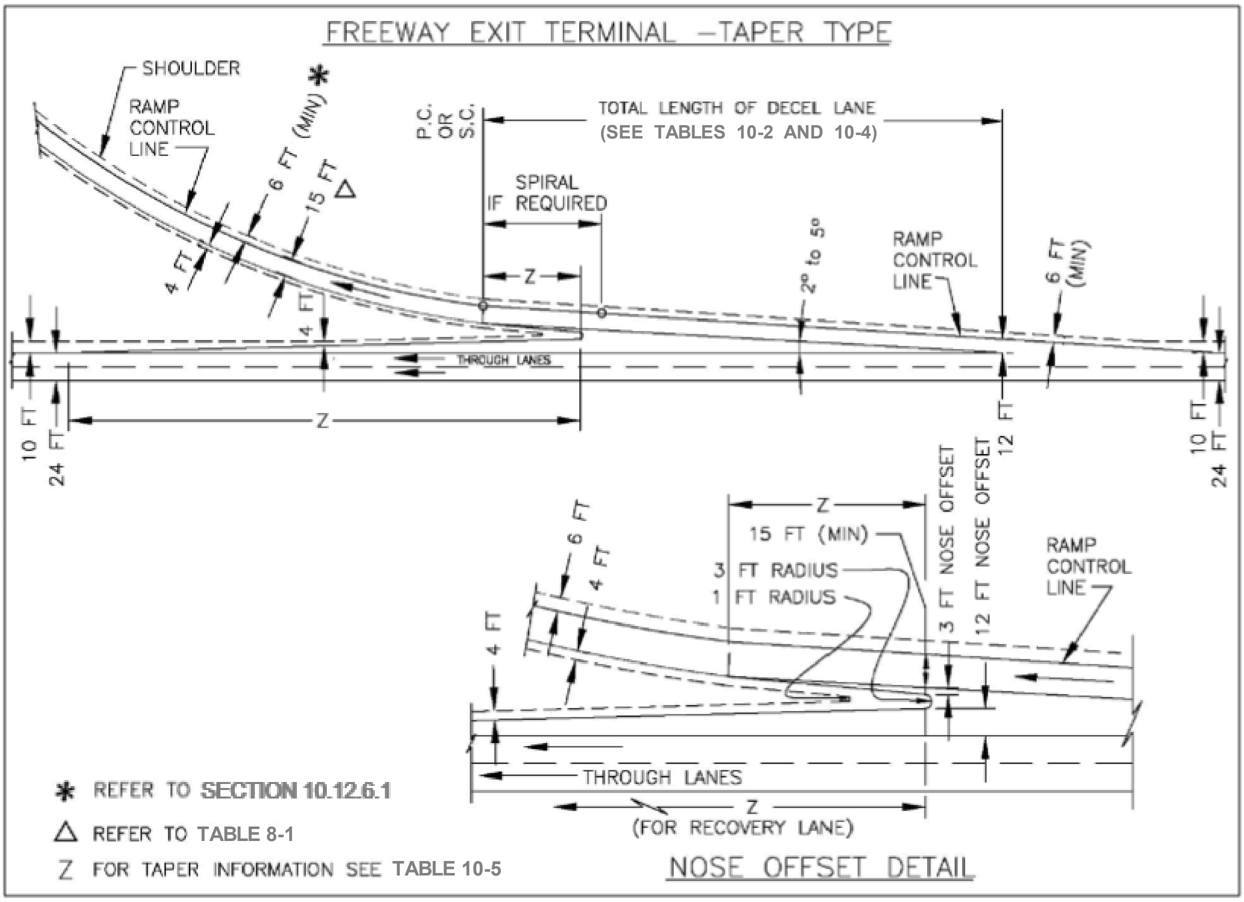


Figure 10-14 Freeway Exit Terminal (Parallel Type Z)

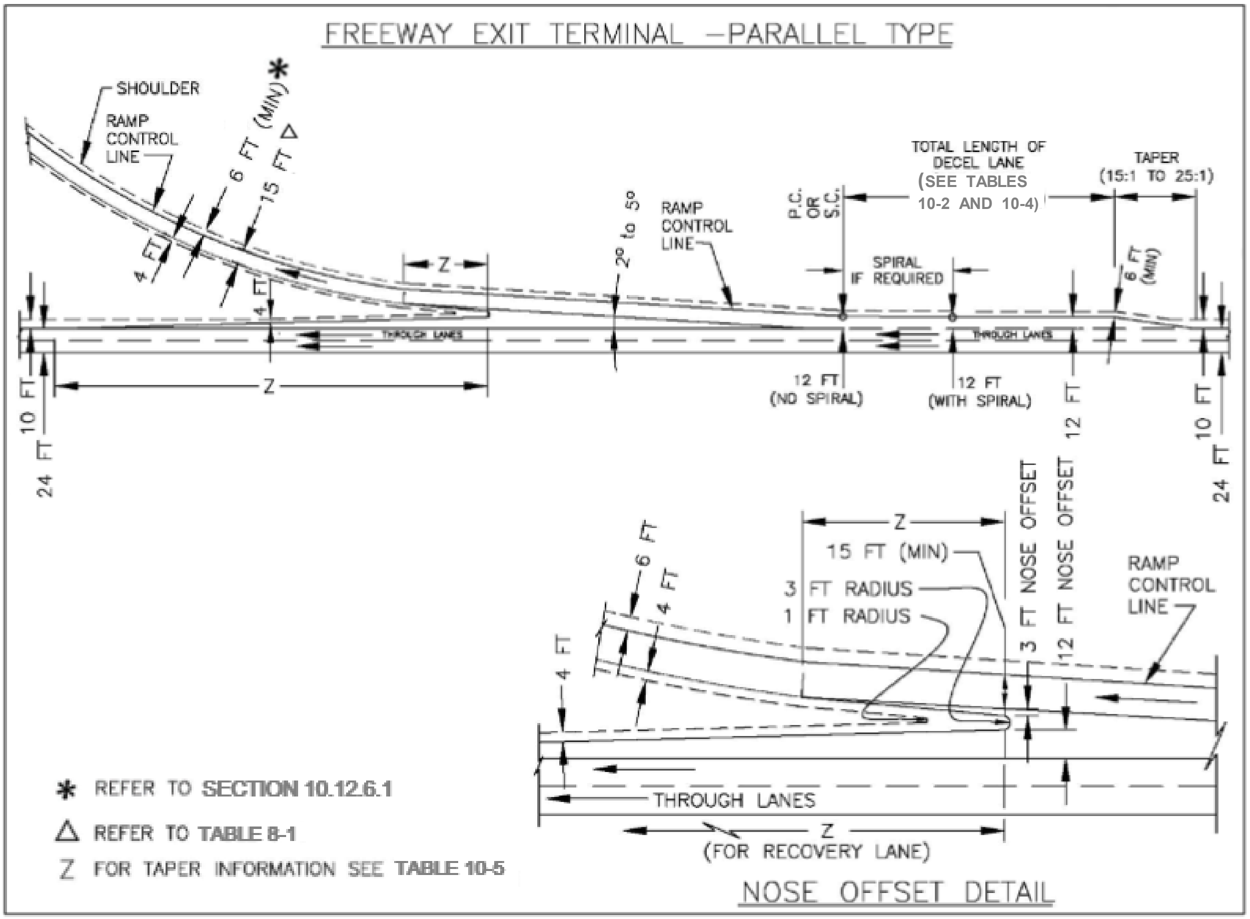


Table 10-5 Minimum Length of Taper Beyond an Offset Nose

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

10.13.2.2 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 accommodates proper superelevation transitions from the curve to the tangent section in the long, merging end area.

10.13.2.3 Parallel Type Entrance

The parallel type entrance is preferred over the taper type entrance. The parallel type has 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.

Desirably, a curve with a radius of 1,000 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, drivers tend to drive directly onto the freeway without using the acceleration lane, which results in an 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 radius of 1,000 feet or more, and the driver has an unobstructed view of traffic on the freeway to the left, a part of the ramp proper may be considered 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 feet, plus the taper, is desirable whenever it is anticipated that the ramp and freeway carry traffic volumes approximately equal to the design capacity of the merging area.

10.13.2.4 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 a driver will exit near the beginning of the added lane, and slow down 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.

10.13.2.5 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 lane. 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 affect a ramp cross slope different from that of the through lane.

10.13.2.6 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. Refer to Figure 10-71 of the 2018 AASHTO GDHS for layout.

10.13.2.7 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. Refer to Figure 10-15A and Figure 10-15B for examples of a single-lane ramp exit transition to two lanes. Contact the Region Traffic Engineer for required storage lengths.

Figure 10-15A Single-Lane Ramp Exit Transition to Two Lanes (Alternate A)

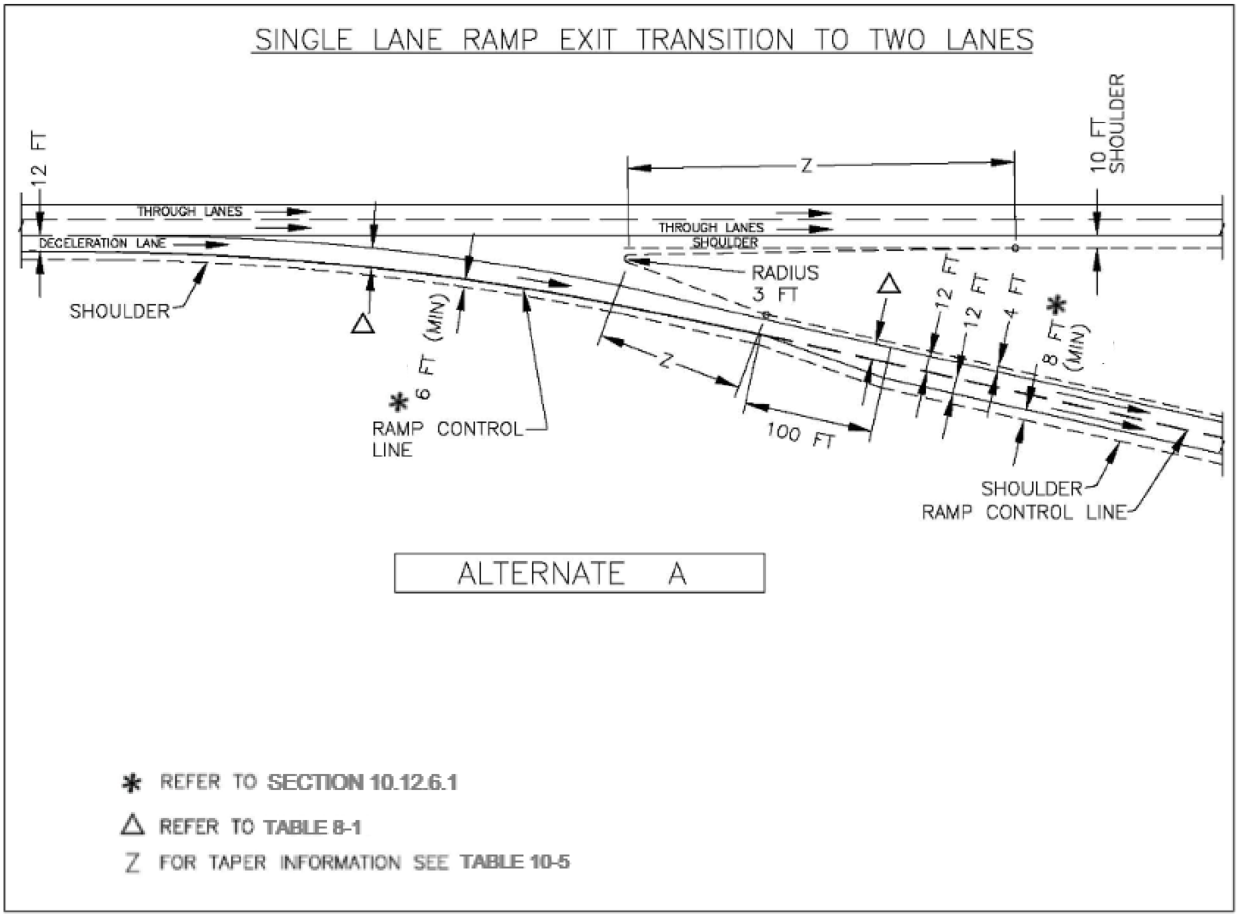
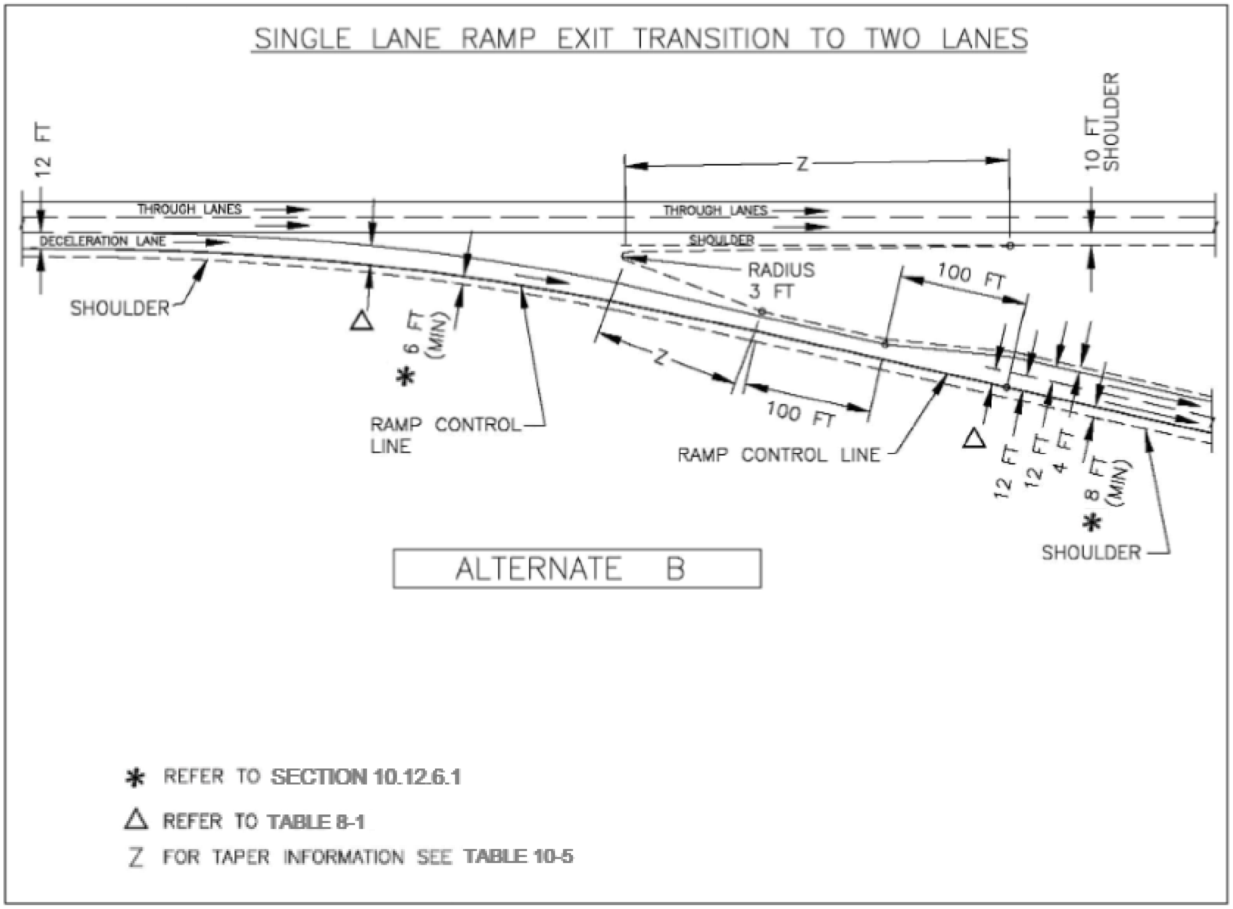


Figure 10-15B Single-Lane Ramp Exit Transition to Two Lanes (Alternate B)



10.13.3 Multilane Free-Flow Terminals

Multilane terminals are required where traffic volumes are too high for single-lane operation. The most common multilane terminals are 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.

10.13.3.1 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 results 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.

10.13.3.2 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. Refer to Figure 10-16B.

Figure 10-16A Two-Lane Entrance or Branch Connection (Taper)

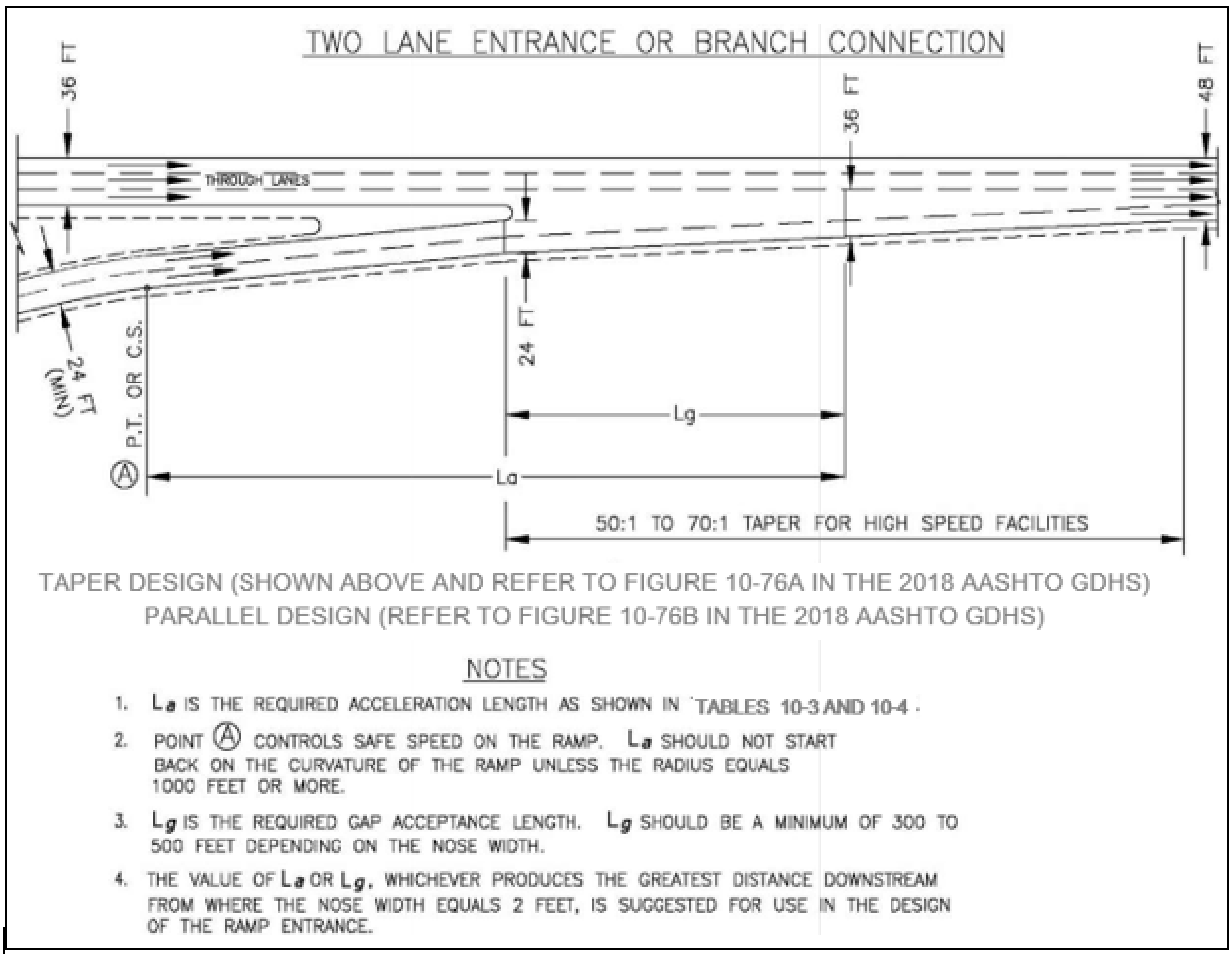
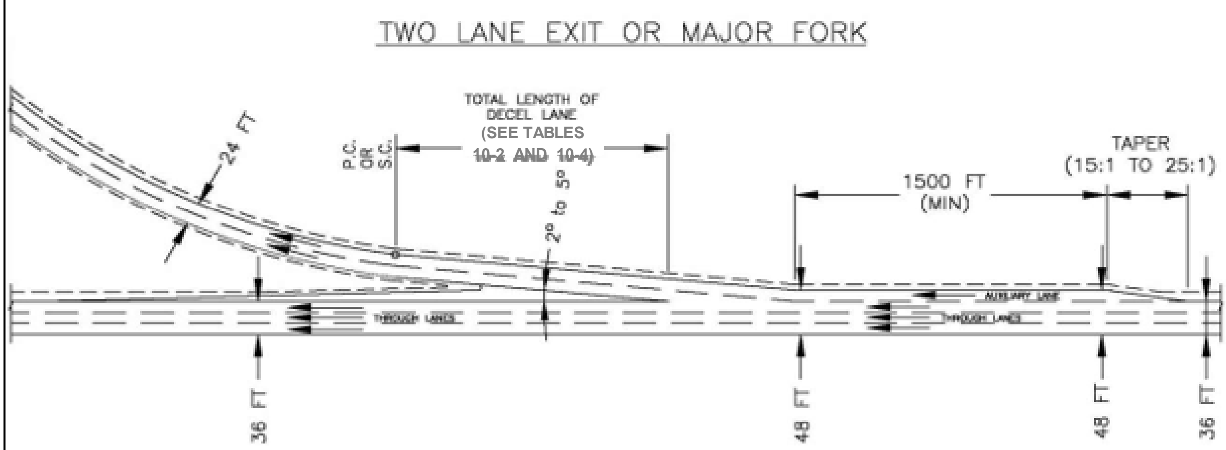


Figure 10-16B Two Lane Exit or Major Fork (Taper)



TAPER DESIGN (SHOWN ABOVE AND REFER TO FIGURE 10-77A IN THE 2018 AASHTO GDHS)
PARALLEL DESIGN (REFER TO FIGURE 10-77B IN THE 2018 AASHTO GDHS)

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 vehicles per hour (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 10-16B.

For two-lane exits, the preferred type is the taper type for the same reasons identified in Section 10.10.12 of this chapter on the single-lane exit. For the parallel-type exit, 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 exit requires a longer distance from beginning of the taper to the exit nose to develop the full capacity of the ramp.

10.13.3.3 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 a major fork 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. Refer to Chapter 10 of the 2018 AASHTO GDHS 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. Refer to Figure 10-76 in the 2018 AASHTO GDHS.

10.14 Multimodal Accommodations

The accommodation of pedestrians and bicycles through urban interchanges should be considered early in the development of interchange configurations. High-density land use near an interchange can generate heavy bicycle and pedestrian movements, resulting in conflicts between vehicles and bicycles or vehicles and pedestrians. Refer to Chapter 13 of this Guide for additional information on considerations for bicycle and pedestrian facilities.

10.15 Other Interchange Design Features

10.15.1 Testing for Ease of Operation

Refer to 2018 AASHTO GDHS Chapter 10.9.7

10.15.2 Managed Lanes and Transit Facilities

Refer to 2018 AASHTO GDHS Chapter 10.9.7

10.15.3 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 meters are 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.

The designer should contact the Region Traffic Engineer for design considerations.

For more information refer to 2018 AASHTO GDHS Chapter 10.9.7

10.15.4 Grading and Landscape Development

Refer to 2018 AASHTO GDHS Chapter 10.9.7

10.15.5 Models

Refer to 2018 AASHTO GDHS Chapter 10.9.7