Chapter 10

INTERCHANGES

SOUTH CAROLINA ROADWAY DESIGN MANUAL

February 2021

SPACER PAGE

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Chapter 10
INTERCHANGES

An interchange is a system of ramps in conjunction with one or more grade separations that provides for the movement of traffic between two or more roadways on different elevation levels. The operational efficiency, capacity, safety and cost of the highway facility are largely dependent upon its design. This chapter provides guidance in the design of interchanges including type, selection, operations, spacing, freeway/ramp terminals, ramps and ramp/crossroad terminals.

10.1 GENERAL

10.1.1 Interchange Warrants

High cost and environmental impact require interchanges be provided only after careful consideration of their merits. Because of the variance in specific site conditions, SCDOT has not adopted specific interchange warrants. When determining the need for an interchange or grade separation, consider the following:

1. Access Control. The following will apply:

   a. Full Access Control. On all fully access-controlled facilities, intersecting crossroads must be terminated, rerouted, grade separated or an interchange provided. The importance of the continuity of the crossroad, the feasibility of an alternative route, traffic volumes, construction costs, environmental impacts, etc., are evaluated to determine which option is most beneficial. Interchanges generally are provided at:

      - all freeway-to-freeway crossings;
      - all major highways, unless determined inappropriate; and
      - other highways based on the anticipated demand for regional access.

   b. Partial Access Control. On facilities with partial access control (expressways), intersections with public roads will be accommodated by an interchange or with an at-grade intersection. Generally, it will be rare that a grade separation or an interchange will be provided where the facilities are not access controlled, topography limits the potential to provide grade separation or costs are prohibitive. However, an interchange may still be a viable option for high-volume intersecting roads when considering Items 2 through 6.

   c. No Access Control. An interchange will rarely be warranted on a facility with no access control. The need for an interchange will be determined on a case-by-case basis emphasizing cost effectiveness, safety and operations. A road-user benefit analysis will generally be required to determine the economic feasibility of an interchange; see Item 4. However, this analysis alone is not a sufficient justification for the provision of an interchange.

2. Safety. In special cases, consider the crash-reduction benefits of an interchange at an existing intersection that exhibits extremely high-crash frequencies or rates.

3. Site Topography. Where access is necessary, the topography may dictate an interchange or a grade separation rather than an intersection.

4. Road-User Benefits. If an analysis reveals that road-user benefits over the service life of the interchange will exceed costs, then an interchange may be considered. The designer must consider all costs including right of way, construction, maintenance and user costs in the analysis. For additional guidance, see the AASHTO publication *User and Non-User Benefit Analysis for Highways*.

5. Reduction of Bottlenecks. Insufficient capacity at the intersection of heavily traveled routes often results in significant congestion on one or all approaches. The inability to provide essential capacity with an at-grade facility may warrant an interchange where development and available right-of-way permit. Even on facilities with partial control of access, the elimination of random signalization contributes greatly to improvement of free-flow characteristics.

6. Traffic Volumes. Although there are no specific traffic volumes that warrant an interchange, consider providing an interchange where the traffic volumes at an intersection are at or near capacity and where other improvements are not practical. Consider providing an interchange where the level of service (LOS) at an intersection is unacceptable and the intersection cannot be redesigned to operate at an acceptable LOS.

10.1.2 New/Reconstructed Interchanges

In general, all new and/or modified access points should be minimized on existing fully access-controlled facilities. Each entrance and exit point on the mainline, including locked gate access (e.g., utility opening), is defined as an access point. A modified access includes changes in an existing interchange configuration although the number of access points may not change.

The Department must demonstrate that an additional access point or revision is required for regional traffic demand and not just to solve local system needs or problems. The Interstate and other freeway facilities, including the interchange crossroad and ramps, should not be allowed to become a part of the local circulation system, but should be maintained to handle regional traffic demands.

SCDOT and FHWA must approve all proposed changes in interchange configurations on the Interstate System, even if the number of access points does not change. See the *Federal Register*, Vol. 74, No. 165, August 27, 2009. The FHWA *Interstate System Access Information Guide* provides guidance on the procedures for documenting these requests.

10.1.3 Grade Separation versus Interchange

Section 17.4.2 discusses the justification for a grade separation and general design considerations. Section 17.4.2 also presents criteria for determining if the major road should pass over or under the crossroad. Once it has been determined to provide a grade-separated crossing, the need for access between the two roadways must be evaluated to determine if an interchange is appropriate. The following lists several guidelines to consider in the evaluation:

1. Functional Classification. Provide an interchange at all freeway-to-freeway crossings. On fully access-controlled facilities, provide an interchange with all major highways, unless

this is determined inappropriate for other reasons (e.g., terrain). Consider providing interchanges to other highways, if practical.

2. Site Conditions. Site conditions that may be adaptable to a grade separation may not always be conducive to an interchange. Restricted right of way, environmental concerns, rugged topography, etc., may restrict the practical use of an interchange.

3. Interchange Spacing. Freeway operations are improved with increased interchange spacing. See Section 10.3.1 for guidance on interchange spacing. If these criteria cannot be met, this may favor the use of a grade separation rather than an interchange.

4. Operations. Grade-separated facilities without ramps will allow traffic to cross the facility. All drivers desiring to turn onto the crossroad must use other locations to make their desired moves. This will often improve the operations of the major facility by concentrating the access to a few strategically placed locations. Concentration of the access movements at specific locations will affect the operation of the interchange.

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10.2 INTERCHANGE TYPES AND SELECTION

10.2.1 General

SCDOT uses five basic interchange types — the diamond, cloverleaf, partial cloverleaf, three-leg and directional. These interchange types, and variations within each type, permit adaptation to traffic needs, available right of way, terrain and cultural features. The following sections discuss these basic interchange types and the design elements for laying out the interchange. Each interchange must be designed to fit the individual site considerations. The final design may be a minor or major modification of one of the basic types or may be a combination of two or more basic types.

Section 10.2.13 discusses the selection of an interchange type.

10.2.2 Conventional Diamond

The conventional diamond is the simplest and most common interchange type. Diamonds include one-way diagonal ramps in each quadrant and two intersections at the crossroad. With proper treatments at the crossroad (e.g., intersection capacity, adequate storage distance between ramps, vertical and horizontal alignment), the diamond is often the best interchange choice where the intersecting road is not access controlled. Figure 10.2-A illustrates a typical diamond interchange without frontage roads. Figure 10.2-B illustrates a typical diamond interchange with frontage roads. Some of the advantages and disadvantages of a conventional diamond include:

Advantages

1. All exits from the mainline occur before reaching the crossroad structure and entrances occur after the structure. This conforms to driver expectancy and therefore minimizes confusion.

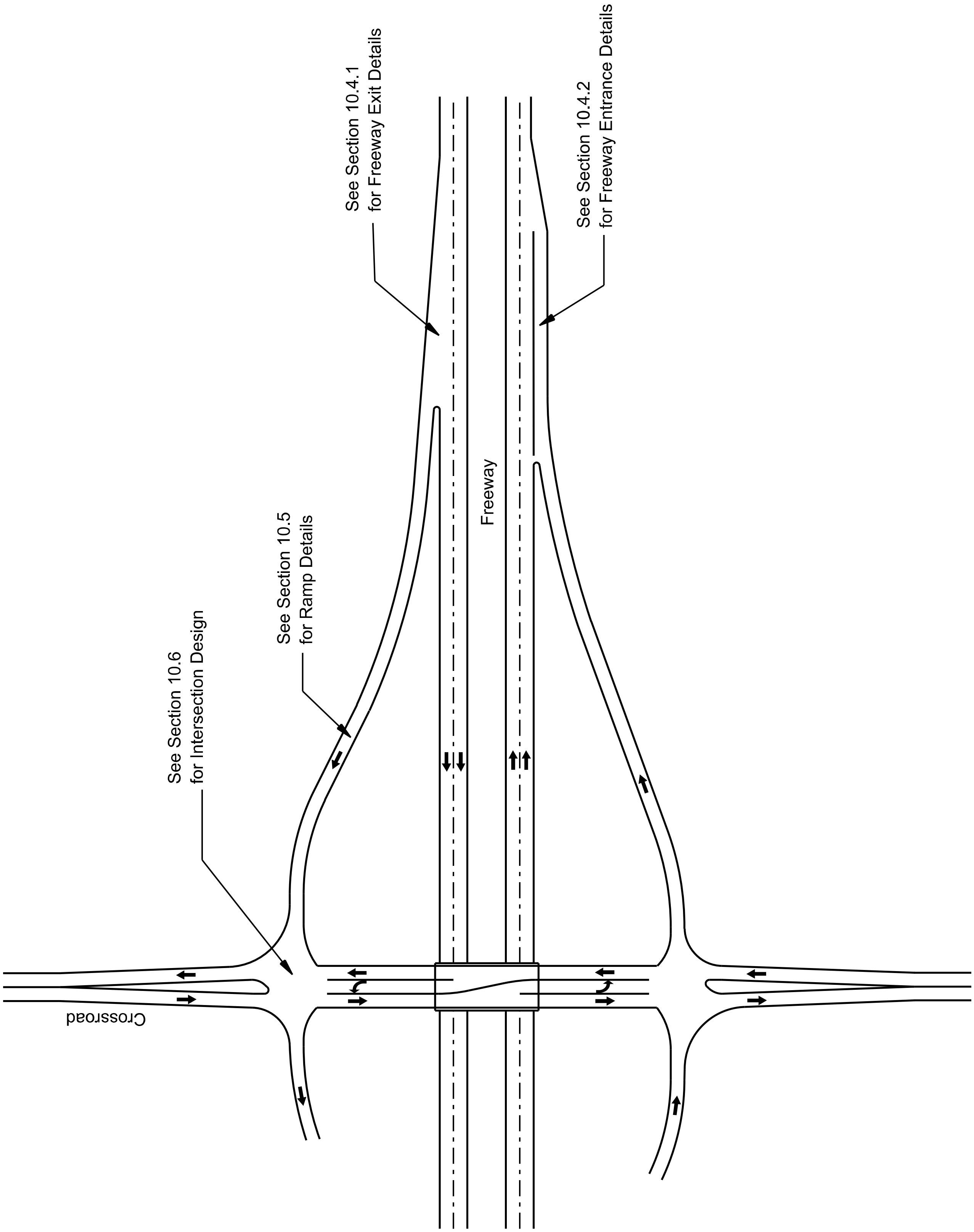
2. All traffic can enter and exit the mainline at relatively high speeds. The operational maneuvers are normally uncomplicated.

3. At the crossroad, adequate sight distance can usually be provided, and the operational maneuvers are consistent with other intersections on the crossroad.

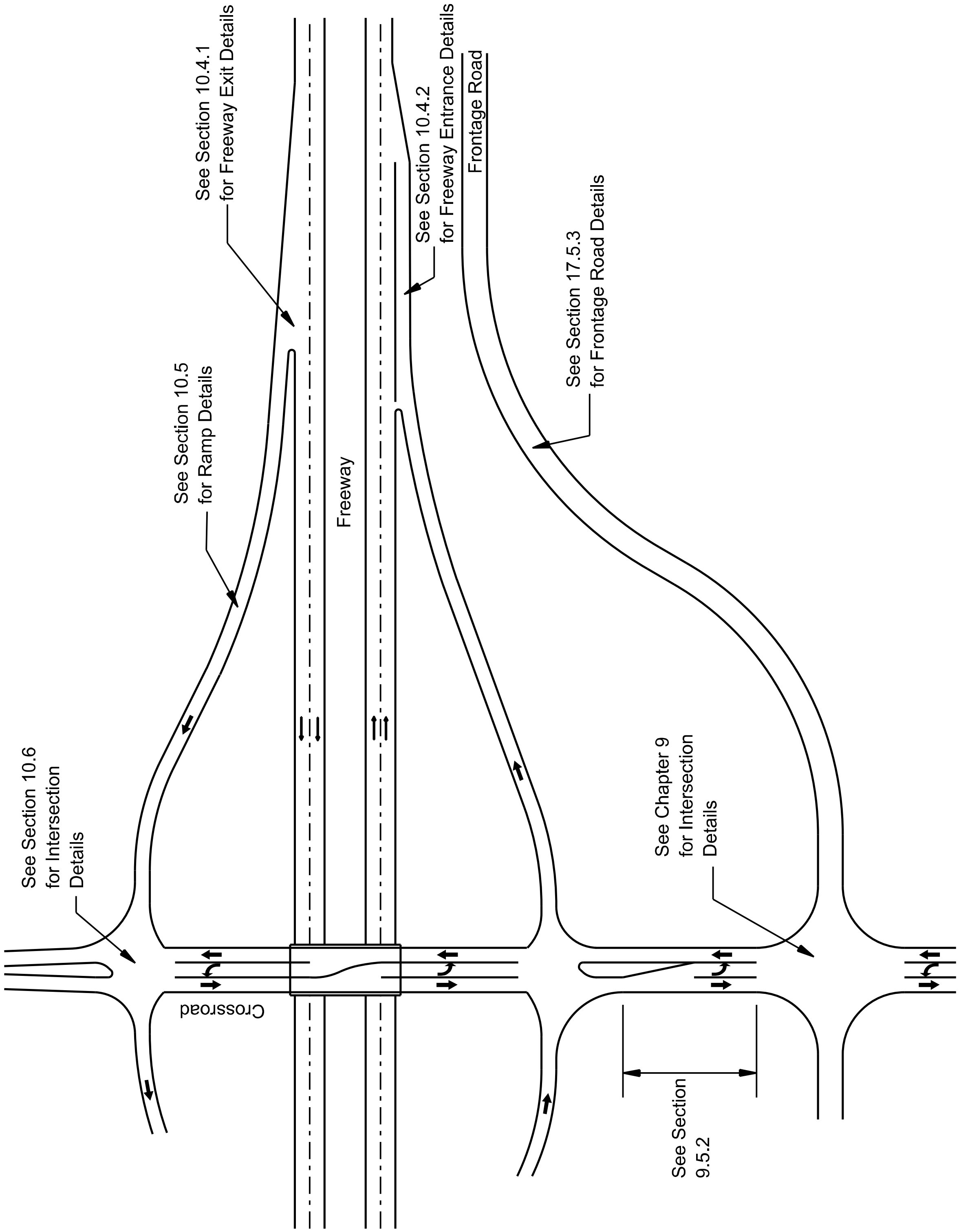
4. The diamond requires less right of way than other interchange types.

5. Their common usage has resulted in a high level of driver familiarity.

6. Typically, it is the least expensive of all interchange types.



DIAMOND INTERCHANGE
(Without Frontage Roads)
Figure 10.2-A



DIAMOND INTERCHANGE
(With Frontage Roads)
Figure 10.2-B

Disadvantages

1. Introduces two at-grade intersections at the crossroad. The designer needs to consider intersection spacing, sight distance, left-turn storage between ramps, signal coordination, etc.

2. Traffic is subject to stop-and-go operations rather than free flow.

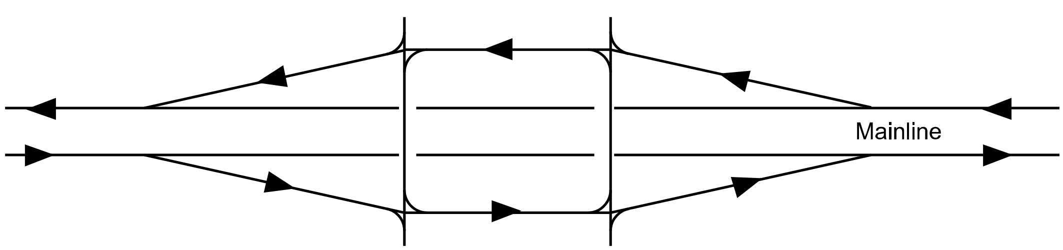
3. In suburban and urban areas, signalization is generally required at the crossroad intersections. These signals may need to be interconnected for progression. Signalization may also produce vehicular platoons entering the freeway, which may cause congestion in the freeway/ramp merge area.

4. A diamond requires right of way in all four quadrants of the interchange.

5. A diamond has a greater potential for wrong-way entry onto the ramps than, for example, a full cloverleaf.

10.2.3 Split Diamond

A variation of the conventional diamond is a split diamond interchange; see Figure 10.2-C. Split diamonds are normally used in urban or suburban areas where the designer desires to provide access to two crossing roadway facilities that are spaced less than one mile apart. Normally, separate interchanges cannot be located within this distance without creating substandard geometric conditions and/or weaving problems without the use of collector-distributor (C-D) roads. It is desirable to make the connecting roadways (between the two crossroads) one-way with control of access. Split diamonds have an undesirable feature in that traffic leaving the freeway cannot return at the same interchange point and continue in the same direction.



SPLIT DIAMOND INTERCHANGE
Figure 10.2-C

10.2.4 Compressed Diamond

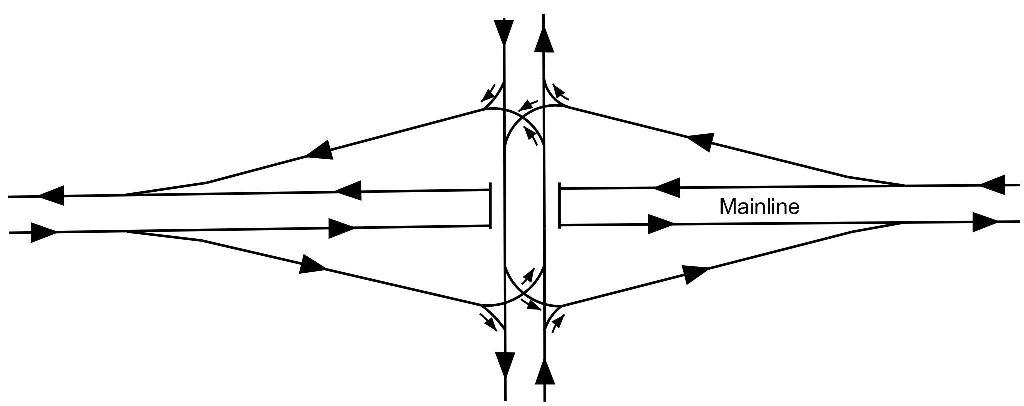
A compressed diamond is similar to the conventional diamond except that the ramp termini on the crossroad are spaced approximately 500 feet to 700 feet apart. Figure 10.2-D presents a schematic of a compressed diamond interchange without frontage roads. This design type is generally only used in urban areas where a diamond interchange is appropriate, but right of way or other environmental features preclude the use of the conventional diamond. Although operationally a compressed diamond is similar to a tight diamond or single-point diamond discussed in Section 10.2.6, they have significant differences. Some of the advantages and disadvantages of the compressed diamond include:

Advantages

1. Generally, less right of way is required than that for a conventional diamond.

2. The open pavement area at the intersection is significantly less than that for a single-point diamond.

3. The grade separation structure is smaller than that for a single-point diamond, retaining walls and/or embankments are less expensive, and construction costs are lower.



COMPRESSED DIAMOND INTERCHANGE
Figure 10.2-D

Disadvantages

1. Left-turn lanes between the ramp termini usually need to be overlapped (i.e., side-by-side opposing left-turn lanes). Consequently, the cross section of the crossroad is generally wider than a conventional diamond.

2. Signal timing and interconnection are necessary to eliminate left-turn queues from overlapping upon each other and causing gridlock.

3. Because of the spacing between intersections and the three-phase signalization, efficient signal coordination is difficult.

4. The length of access control on the crossroad may be more extensive than for a conventional diamond.

5. A diamond has a greater potential for wrong-way entry onto the ramp than a regular diamond.

6. There is the potential for the left turn lane queue to block ramp access.

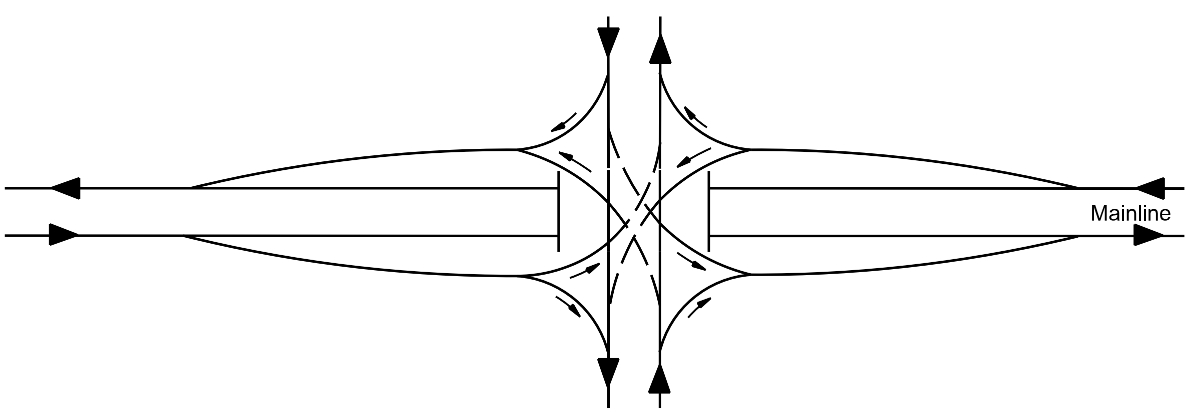
10.2.5 Tight-Urban Diamond

The tight-urban diamond interchange is similar to the compressed diamond except that the spacing between the two intersections is only 250 feet to 350 feet apart. In addition, only one signal controller is required for the tight-urban diamond versus two for the compressed diamond. In addition to the advantages of the compressed diamond, the tight-urban diamond intersections operate as a single intersection for signal control. The signal timing and phasing scheme typically precludes the need for storage of vehicles between the two intersections. All vehicles are stored external to the two intersections on the cross street and on the exit ramp approaches.

The operations and geometrics along the freeway approach for tight-urban diamond and single point diamond interchanges are essentially identical. The advantages of the tight-urban diamond compared to single-point diamond are that structural costs are typically lower for the tight-urban diamond and that pedestrians, bicyclists and frontage roads can be easier accommodated with the tight-urban diamond.

10.2.6 Single-Point Diamond

The single-point diamond interchange (SPDI) is a variation of a tight-urban diamond except instead of two intersections there is only one for the SPDI. This interchange offers improved traffic-carrying capabilities, safer operations and reduced right of way needs under certain conditions when compared with other interchange configurations. The distinguishing feature of this interchange is the convergence of all through and left-turning movements into a single, large signalized intersection area. Figure 10.2-E illustrates a schematic of a SPDI. Some of its advantages and disadvantages include:



SINGLE-POINT DIAMOND INTERCHANGE
Figure 10.2-E

<u>Advantages</u>

1. The SPDI only requires one intersection instead of two intersections at a typical diamond.

2. It allows for better traffic signal progression on the crossroad.

3. The SPDI can increase interchange capacity and alleviate storage problems from two closely spaced intersections on the crossroad.

4. Opposing left turns operate to the left of each other so that their paths do not cross each other.

5. Less right of way is required than any other interchange type.

6. At the intersection of the ramps with the crossroad, the design typically includes flatter curves for turning radii, which allows left turns to be completed at higher speeds.

<u>Disadvantages</u>

1. Special pavement markings and a centrally located diamond-shaped island are required to guide the left-turning drivers through the intersection.

2. There is a significantly wider pavement area for pedestrians to cross, and the SPDI may create greater delays in traffic when compared to the conventional diamond.

3. Because of wide pavement areas, it requires longer signal clearance times.

4. The SPDI has a higher cost than the conventional or compressed diamond because of the need for a long, single-span structure and the need for retaining walls or reinforced earth walls along the mainline.

5. Where the mainline is over a crossroad, lighting is required under the structure.

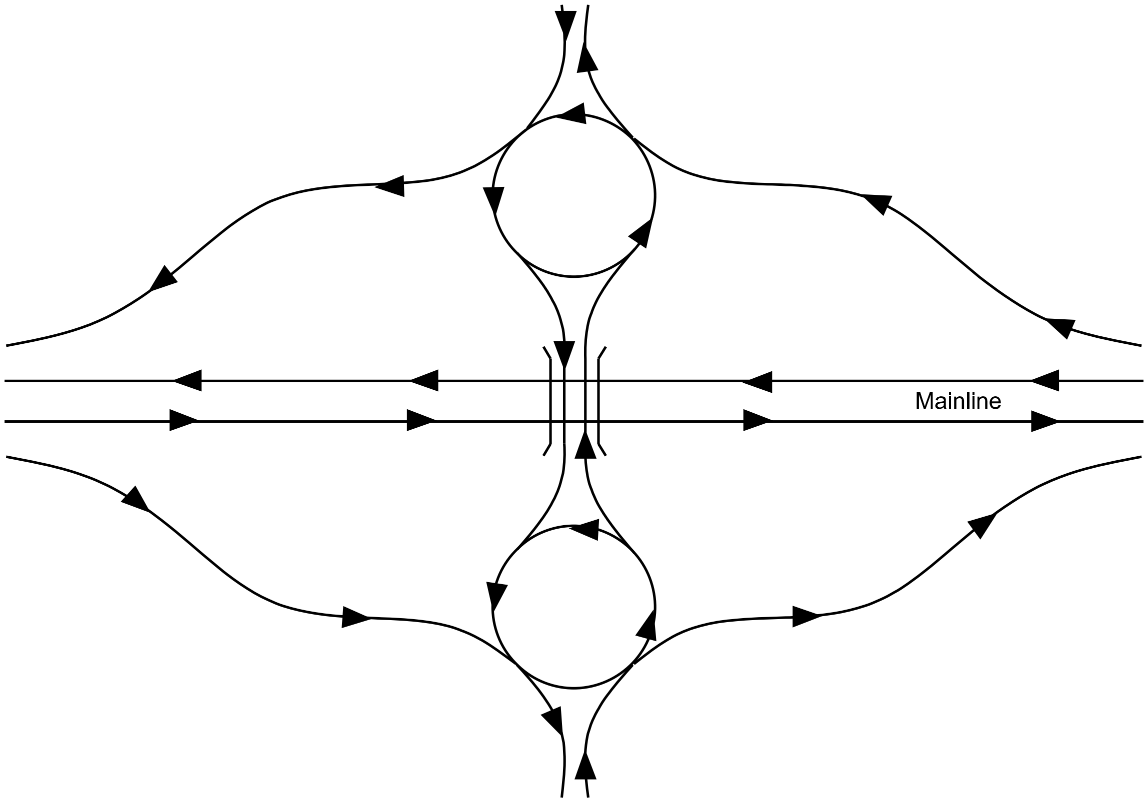
See NCHRP Report 345, *Single-Point Urban Interchange Design* and *Operational Analysis* for complete design details of a SPDI.

10.2.7 Double Roundabout Diamond

A double roundabout diamond, also called a dumbbell or dog-bone diamond interchange, contains roundabouts at each crossroad ramp. Figure 10.2-F presents a schematic of a double roundabout diamond interchange. Free-flow through movements are provided by using two-single or multi-lane roundabouts on the cross street to accommodate left and right turns and all movements on the cross street. The design provides a narrower bridge (no storage turn lanes) and the elimination of signal control at the interchange. Some of the advantages and disadvantages of the double roundabout diamond include:

<u>Advantages</u>

1. The roundabouts allow almost continuous flow; reduced delay for ramp traffic.



DOUBLE ROUNDABOUT DIAMOND INTERCHANGE
Figure 10.2-F

2. Signal coordination and progression issues are eliminated between the two ramp terminals.

3. The overpass bridge can be narrower, because left-turn lanes are eliminated from the bridge.

4. They are an easy-to-build step up from the diamond junction.

5. Reduces the number of conflict points.

Disadvantages

1. Profile grades may need to be 3 percent or flatter on all approaches to ensure adequate sight distance.

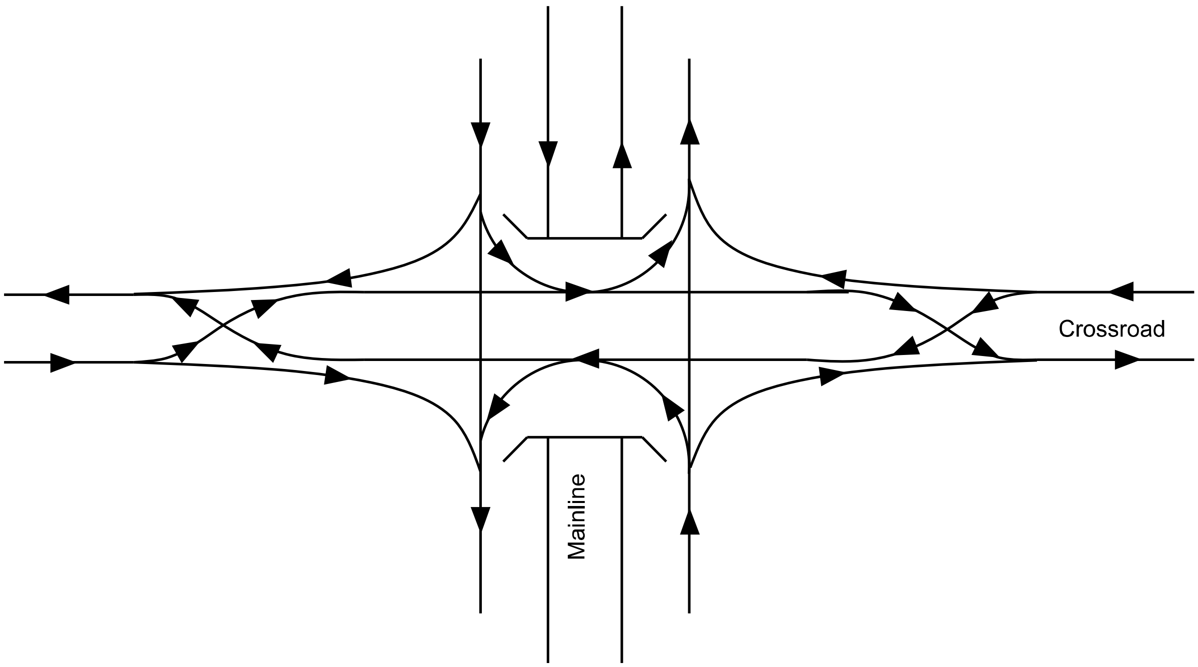
2. May not have sufficient capacity for high-volume intersections.

3. Pedestrian and bicycle movements may be hindered.

10.2.8 Diverging Diamond Interchange

The diverging diamond interchange transposes the opposing directions of crossroad traffic through the interchange, see Figure 10.2-G. This configuration allows all turning traffic from or onto the freeway to turn without crossing in front of the opposing traffic. By doing this, the two traffic signals only need to operate as a two-phase signal control instead of three-phase control for other diamond interchanges.

The primary reason for considering the diverging diamond interchange is due to its increased capacity and/or reduced lane requirements versus other diamond interchange configurations. It is only applicable in urban or suburban areas where operating speeds are no more than 40 to 50 miles per hour. Special consideration must be given to the design to ensure the interchange operates properly (e.g., flared approaches, channelization).



DIVERGING DIAMOND INTERCHANGE
Figure 10.2-G

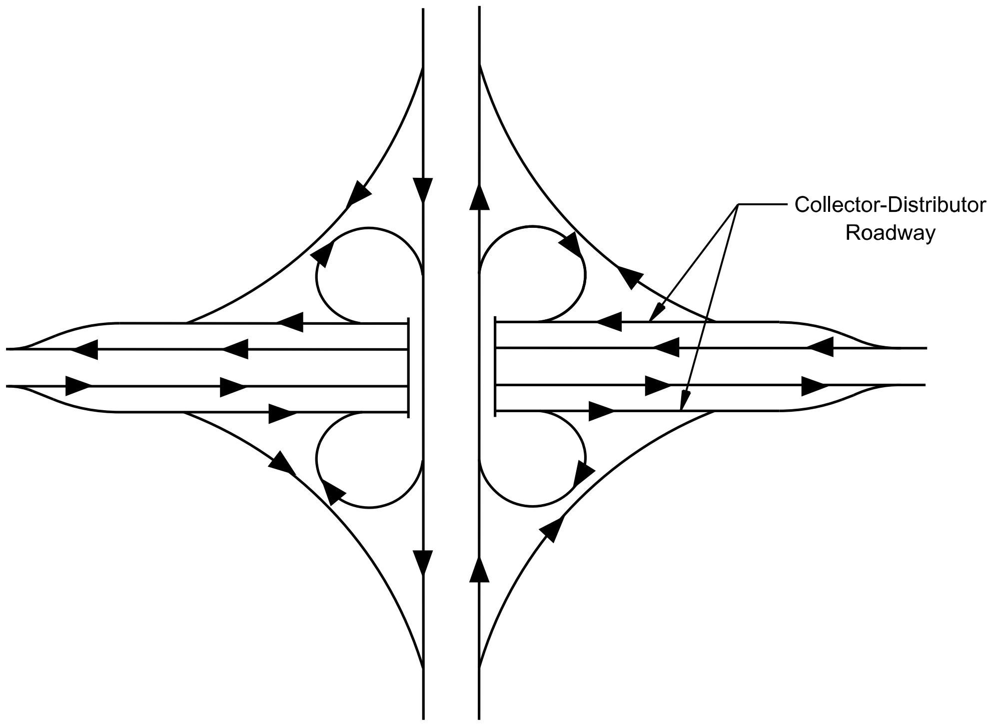
10.2.9 Full Cloverleafs

Cloverleaf interchanges are used at four-leg intersections and employ loop ramps to accommodate left-turn movements. Cloverleaf interchanges without collector-distributor (C-D) roadways are seldom considered a viable option. Full cloverleaf interchanges are those with loops in all four quadrants; all others are partial cloverleafs; see Section 10.2.10.

Where two access-controlled highways intersect, a full cloverleaf is the minimum type of interchange design that will suffice. In addition, they also may be used at the intersection of other multilane arterials to accommodate large volumes of traffic.

The operation of a cloverleaf with high weaving volumes is greatly improved through the addition of collector-distributor (C-D) roadways; see Section 10.3.8. The C-D roadways may be advantageous in suburban areas because of the need for smaller loops. This may reduce the amount of right of way acquisition necessary for the development of the interchange. Although right of way requirements may be reduced, overall costs usually increase due to longer and wider structures and additional pavement costs.

Figure 10.2-H provides a typical example of a full cloverleaf with C-D roads.



CLOVERLEAF INTERCHANGE WITH C-D ROADWAYS
Figure 10.2-H

Some of the advantages and disadvantages of full cloverleafs include:

<u>Advantages</u>

1. Full cloverleafs eliminate all vehicular stops through the use of free-flow terminals, and they provide continuous free-flow operation on both intersecting highways.

2. Full cloverleafs eliminate all at-grade intersections, eliminate left turns across traffic and, therefore, eliminate the need for traffic signals.

<u>Disadvantages</u>

1. Because of the geometric design of loops, full cloverleafs require large amounts of right of way.

2. They are typically more expensive than diamond interchanges due to considerably lengthier ramps, wider structures and, if provided, the additional cost of C-D roads.

3. The loops in cloverleafs result in a greater travel distance for left-turning vehicles than do diamonds, and the speeds on the ramps are generally slower.

4. Exit and entrance terminals are located before and after the crossroad structure, which require additional signing to guide motorists.

5. Weaving sections between loop ramps must be long enough to provide for satisfactory traffic operations.

6. Where the crossroad is an expressway or other multilane highway, a considerable length of access control distance is needed along the crossroad to the first point of access.

7. Pedestrian movements are difficult to accommodate.

Operational experience with full-cloverleaf interchanges has yielded several observations on their design. Subject to a detailed analysis on a site-by-site basis, the following generally characterize the design of cloverleafs:

1. <u>Design Speed Impacts</u>. For an increase in design speed, there will be an increase in travel distance and required right of way.

2. <u>Loop Radii</u>. Design of loop radii is highly dependent on the relative design speed of the two crossing roadways. Consistency with the exit speed on the upstream end and entrance speed requirements on the downstream end are essential.

3. <u>Loop Geometry</u>. Circular curve loop ramps are desirable geometrically because speeds and travel paths tend to be more constant and uniform. However, compound curves are often used as site conditions dictate. Transition of the design speed from curve to curve into and out of the loop is critical.

4. <u>Loop Capacity</u>. Expected design capacities for single-lane loops range from 800 to 1200 vehicles per hour. The higher volumes are generally only achievable where the design speed is 30 miles per hour or higher and few trucks use the loop.

5. <u>Weaving Volumes</u>. An auxiliary lane is typically provided between successive entrance/exit loops within the interior of a cloverleaf interchange. This produces a weaving section between the mainline and entering/exiting traffic. Where the total volume on the two successive ramps reaches approximately 1000 vehicles per hour, there may be a significant reduction of the through travel speed and level of service. Where this occurs, consider providing collector-distributor roadways.

6. <u>Weaving Lengths</u>. The minimum weaving length between the exit and entrance gores of loops on new cloverleaf interchanges without collector-distributor roadways or those undergoing major reconstruction should be at least 1000 feet or the distance determined by a capacity analysis, whichever is greater.

7. <u>Collector-Distributor Roadways</u>. The consideration of collector-distributor roadways should be an integral part of cloverleaf design. They deploy the exit in advance of the crossroad and encourage a lower speed weaving area, which is easier to match with the loop design.

10.2.10 <u>Partial Cloverleafs</u>

10.2.10.1 General

Partial cloverleaf interchanges are those with loops in one, two or three quadrants. See Figure 10.2-I. Several of the advantages and disadvantages for full cloverleafs also apply to partial cloverleafs (e.g., geometric restriction of loops). However, some specific advantages of partial cloverleafs include:

1. Partial cloverleafs provide access where one or more quadrants present adverse right of way and/or topographic problems that preclude a typical diamond interchange.

2. Partial cloverleafs may accommodate heavy left-turn traffic by means of a loop and thereby improve capacity, operations and safety.

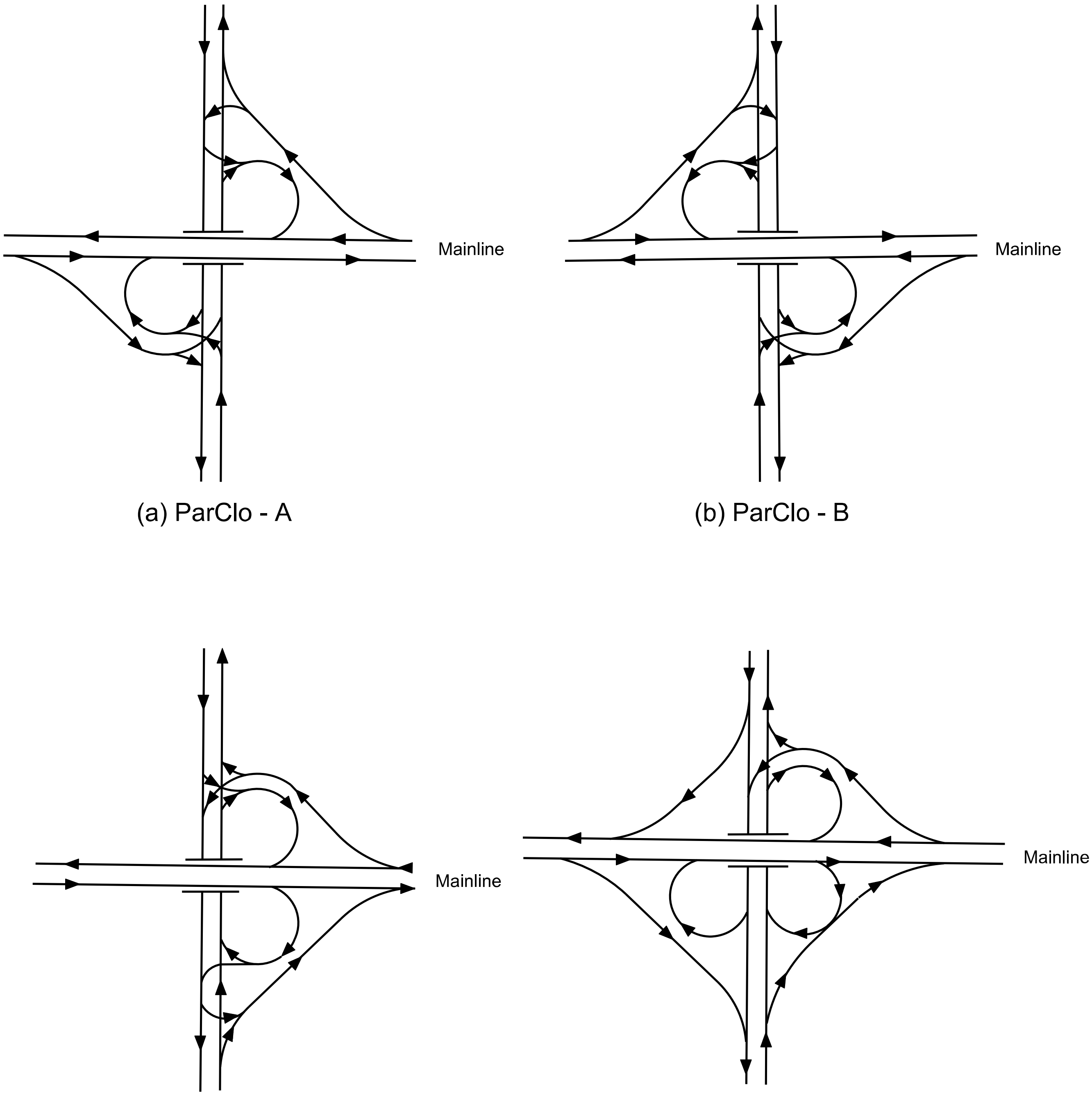
3. Depending upon site conditions, partial cloverleafs may offer the opportunity to eliminate or increase weaving distances.

10.2.10.2 ParClo-A Interchanges

The ParClo-A interchange is a high capacity service interchange appropriate in suburban areas. The two loop ramps are in opposite quadrants and accommodate the left-turning traffic from the cross street onto the freeway; see Figure 10.2-I(a). The term ParClo-A refers to the location of the loop ramps in relation to the driver approaching the interchange (cross street) is in advance of the cross street. This is true in either direction of travel on freeway. If signalized, the two intersections on the cross street are dependent on the radii of the loop ramps. With 150 feet radii loop ramps, the intersections are approximately 700 feet apart. These two-phase signalized intersections can be coordinated for efficient operation.

10.2.10.3 ParClo-B Interchanges

The ParClo-B interchange also has two loop ramps in opposite, but different quadrants than the ParClo-A interchange; see Figure 10.2-I(b). For the driver on the freeway approaching ParClo-B interchange, the loop ramp on the right side of the freeway is beyond the cross street. Drivers exiting the freeway make left turns through the loop ramps. The ParClo-B is appropriate in suburban areas where a freeway interchanges with an arterial street. Although it has similar capacity as the ParClo-A, it has different design and operational characteristics.



(a) ParClo - A

(b) ParClo - B

(c) ParClo - AB

(d) ParClo - Four Quadrant

PARTIAL CLOVERLEAF INTERCHANGES
Figure 10.2-I

10.2.11 Three-Leg Interchanges

Three-leg interchanges are also known as trumpet (or jug handle), Y-type or T type interchanges. Figure 10.2-J illustrates examples of three-leg interchanges with different methods of providing the turning movements.

10.2.11.1 Trumpet (or Jug Handle)

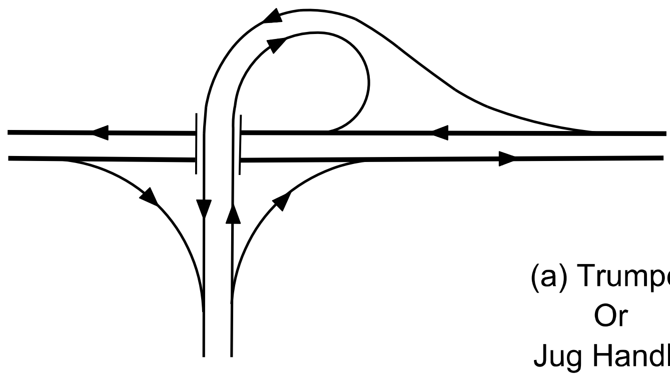
The trumpet (or jug handle) type is shown in Figure 10.2-J(a) where two of the turning movements are accommodated with direct connections, one with a semi-direct connection and one movement by a loop ramp. See Section 10.2.12 for a description of directional connection ramps. This most widely used three-leg interchange has a single (or twin) structure. The criteria for choosing the orientation of ramps depend on the expected traffic volumes or the left-turning movements. The semi-direct connection, sometimes called a jug handle, is used when the highest volume is the left-turning movement, and the lesser volume is carried by the lower capacity loop ramp. Semi-direct connections will handle traffic volumes in the range of 1200 to 1500 vehicles per hour, while loop ramps are limited to 800 to 1200 vehicles per hour. The trumpet interchange is often adaptable to the intersection of a major highway with a principal arterial or freeway.

10.2.11.2 Y-Type

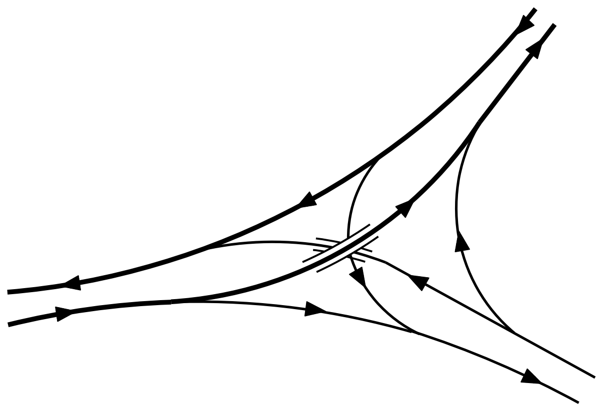
The Y-Type is the costliest of three-leg interchanges, but highly efficient for traffic operations; see Figure 10.2-J(b). This interchange type has two or more structures or one three-level structure and provides high-capacity directional traffic movements (approximately 1200 to 1500 vehicles per hour per ramp) without loops. However, the use of this interchange can only be justified by the requirement to provide for directional traffic movements in excess of 800 vehicles per hour. This type of interchange is most common where two major roads join and continue as a single roadway. In addition, this type may be considered where right of way is limited.

10.2.11.3 T-Type

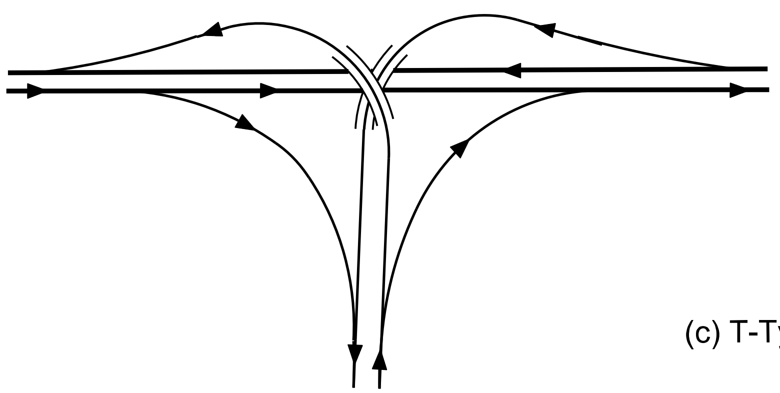
The T-type interchange is shown in Figure 10.2-J(c) where two of the turning movements are accommodated with direct connections and two with semi-direct connections. The criteria for choosing this type of interchange is the high volume of left-turn movements. Semi-direct connections are used when the highest volumes are left-turning movements. Semi-direct connections will handle traffic volumes in the range of 1200 to 1500 vehicles per hour, while loop ramps are limited to 800 to 1200 vehicles per hour. The T-type interchange is often adaptable to the intersection of a major highway with a principal arterial or freeway. Also, this type of interchange may be considered where the right of way is limited.



(a) Trumpet
Or
Jug Handle



(b) Y-Type



(c) T-Type

THREE-LEG INTERCHANGES
Figure 10.2-J

10.2.12 Directional Interchanges

The following definitions apply to directional interchanges:

1. Direct Connection. A ramp that does not deviate greatly from the intended direction of travel; see Figure 10.2-K(a), (b) and (c).

2. Semi-Direct Connection. A ramp where the driver first exits to the right, heading away from the intended direction of travel, gradually reversing, and then passing around other interchange ramps before entering the other road; see Figure 10.2-K(b) and (c).

Direct and semi-direct connections are used for heavy left-turn movements to reduce travel distance, increase travel speed and capacity, and eliminate weaving. These types of connections allow an interchange to operate at a better level of service than is possible with loops. The capacity of a direct or semi-direct connection is approximately 1200 to 1500 vehicles per hour per ramp lane. Exits and entrances on the left should be avoided.

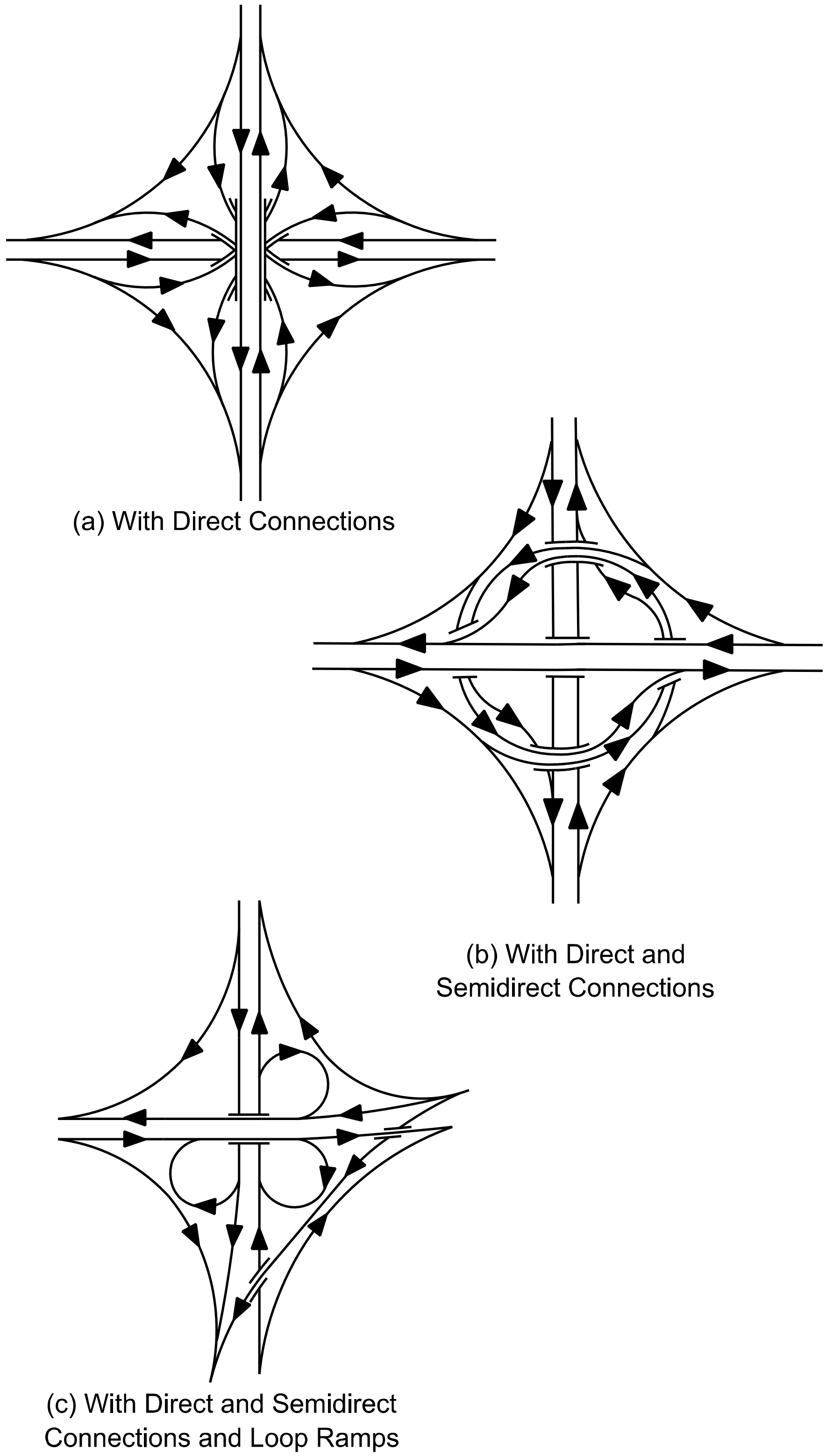
Directional interchanges are most often provided in urban or suburban areas at freeway-to-freeway or freeway-to-arterial intersections. In rural areas, there is generally an insufficient traffic volume to justify the use of direct or semi-direct connections in all quadrants. A directional interchange provides the highest possible capacity and level of service, but it is often costly to construct due to the number of structures required and amount of embankment. Because motorists perceive that higher operating speeds are possible on directional roadways, the alignment of these facilities should be as free flowing as practical.

10.2.13 Selection

10.2.13.1 Evaluation Factors

The designer should evaluate the following factors when selecting an interchange type:

- compatibility with the highway system and functional classification of the intersecting highway;
- route continuity and uniformity with adjacent interchanges;
- level of service for each interchange element (e.g., freeway/ramp junction, ramp proper, ramp/crossroad terminal);
- operational and safety considerations (e.g., signing);
- availability of access control along the crossroad;
- road-user impacts (e.g., travel distance and time, convenience, comfort);
- constructability/maintenance of traffic;
- driver expectancy;
- topography and geometric design;
- right-of-way impacts and availability, construction and maintenance costs and potential for stage construction;
- accommodation of pedestrians and bicyclists on crossroad;
- environmental impacts; and
- potential growth of surrounding area.



DIRECTIONAL INTERCHANGES
Figure 10.2-K

10.2.13.2 General Considerations

The designer should consider the following general factors that will influence the selection of an interchange type:

1. Basic Types. A freeway interchange will be one of two basic types. A systems interchange will connect a freeway to a freeway; a service interchange will connect a freeway to a lesser facility.

2. Freeways. For system interchanges of two fully access-controlled facilities, the minimum design will be a full cloverleaf interchange with collector-distributor roads. Where traffic volumes are significant, a directional interchange may be the most appropriate interchange type.

3. Movements. All interchanges should provide for all movements, even when the anticipated turning volume is low. An omitted maneuver may be a point of confusion to those drivers searching for the exit or entrance. In addition, unanticipated future developments may increase the demand for that maneuver.

4. Capacity. The need for loop ramps or other free-flowing ramps may depend upon the capacity of the ramp termini to adequately accommodate the turning traffic. Conduct a traffic analysis to determine if the ramp termini will be adequate and to determine the appropriate number of approach lanes on the crossroad and ramps.

5. Rural. In rural areas where interchanges occur relatively infrequently, the design can normally be selected strictly on the basis of service demand and analyzed as a separate unit. For most locations, the diamond or partial cloverleaf interchanges are the most appropriate interchange types.

6. Urban. In urban areas the selection of the interchange type is much more complex. In addition to the criteria above, the designer should consider the following factors:

    a. Right of Way. Right of way, in general, is more restricted in urban areas, thereby limiting the available interchange types. This may eliminate the use of a full cloverleaf. In highly restricted locations, the use of a tight diamond or single-point diamond interchange may be the only practical option.

    b. Spacing. Closely spaced interchanges may be influenced directly by the preceding or following interchange such that additional traffic lanes may be required to satisfy capacity, weaving and lane balance.

    c. High-Traffic Volumes. Ramps with high volumes may require free-flowing ramp crossroad terminals to adequately accommodate the turning traffic. High-traffic volumes may also cause problems with weaving sections. To accommodate these concerns may require partial cloverleafs.

    d. Urban System. Evaluate all interchanges along an urban route on a system-wide basis rather than on an individual basis. This will require a corridor analysis reviewing several alternative interchange layouts and types.

e. Crossroads. A thorough study of the crossroad is necessary to determine its potential for accommodating the increased volume of traffic that an interchange will discharge. The ability of the crossroad to receive and discharge traffic from the freeway has considerable bearing on the interchange geometrics (e.g., using loops to eliminate left-hand turns from a conventional diamond).

f. Environmental/Community Factors. Environmental concerns or community opposition to a particular interchange design may impact the selection of an interchange type. For example, a single-point diamond interchange or compressed diamond will require less right of way than a partial cloverleaf.

7. Turning Traffic. Where turning traffic is significant, the ramp profiles are best fitted when the major road is at the lower level. The ramp grades then assist turning vehicles to decelerate as they leave the major highway and to accelerate as they approach it. In addition, for diamond interchanges, the ramp terminal is visible to drivers as they leave the major highway.

10.2.13.3 Capacity (Traffic Volume) Considerations

Interchange type selection, in part, is based upon providing the capacity and level of service that is consistent with the type of highway (major vs. minor) and the anticipated traffic movement between the two facilities. In the hierarchy of interchanges, diamonds provide the lowest in traffic capacity followed in ascending order by partial cloverleafs, cloverleafs and directional. C-D roads can be used with all of these interchange types as necessary to enhance traffic flow and safety, and reduce weaving problems. They are particularly effective in urban freeway design where spacing between interchanges is less than the desired minimum. Figure 10.2-L provides capacity guidelines for initially determining the number of freeway lanes and type of interchange ramps. The final design must be checked with a traffic analysis.

10.2.13.4 Summary

The following presents a summary of the general application of the basic interchange types to general site conditions:

1. Diamond Interchanges. Where left-turning movements are low to and from the major highway, diamond interchanges are normally adequate. The capacity of diamond interchanges is limited by the capacity of the at-grade ramp/crossroad intersection. As left-turn movements at diamond interchanges increase, additional lanes and/or traffic signals at one or both ramp/crossroad intersections may be required.

2. Partial Cloverleaf Interchanges. Where one or more left-turn volumes are significant (500 vehicles per hour or more), loop ramp(s) may be added to provide the partial cloverleaf design to allow for continuous flow. Partial cloverleaf interchanges are an effective compromise between diamonds and full cloverleafs. Their usage may be dictated by limited right of way in one or more quadrants or by a need to maintain continuous flow on those left-turn movements that are disproportionately high or by operational limitations of the crossroad that require the elimination of certain crossing maneuvers.

3. Full Cloverleaf Interchanges with C-D roads. Full cloverleafs are the minimum type interchange used at the intersection of two fully access-controlled highways. Limiting factors in the selection of cloverleaf interchanges are the availability of large amounts of right of way (in the vicinity of 20 acres per quadrant) and the capacity for handling weaving at consecutive entrance/exit terminals. Collector-distributor roads may be necessary to eliminate the weaving problems along the mainline roadways.

4. Directional Interchanges. Interchanges with direct ramp or semi-direct ramp connections for all left-turning movements are often used at intersections of two high-volume freeways having large and nearly equal traffic volumes interchanging between the two facilities.

5. Other/Hybrid Interchanges. Other interchanges or combinations of the interchanges described above may be considered.

Segment(1)		**Traffic Volume (veh/h)(1)**	**Guidance**
Freeway(2)	Urban	0 – 5010	Two lanes in each direction
		5010 – 6680	Three lanes in each direction
		> 6680	Four lanes in each direction
	Rural		See *Highway Capacity Manual*.
Ramps		0 – 300	Single lane ramp
		300 – 500	Dual lefts OR free-flow rights
		500 – 800	Loop rams, semi-direct free flow
		800 -1200	Direct ramp, free flowing, no weaving
		> 1200	Two-lane entrance/exit ramps at freeway junction
Arterials		0 – 2220	Two lanes in each direction
		2220 – 3340	Three lanes in each direction

Notes: (1) All values are for general guidance. A traffic analysis is required for final design based on actual design conditions.

(2) Contact the SCDOT Planning Office for existing LOS for freeway segments.

INTERCHANGE CAPACITY GUIDELINES
Figure 10.2-L

10.3 INTERCHANGE SYSTEM OPERATIONS AND DESIGN

Part I “Roadway Design Elements” and Chapter 17 “Freeways” of the *SCDOT Roadway Design Manual* present Department criteria for several operational and geometric design elements (e.g., design speed, horizontal/vertical alignment, cross sections, frontage roads, over/under determinations, freeway lane drops) that apply to an interchange. This section discusses operational and design considerations that apply to the overall interchange system and, in many cases, are unique to interchange design.

10.3.1 Interchange Spacing

Interchange spacing is based on the demand for access, adequate distance to provide for signing and weaving and adequate distance to permit the adjacent interchanges to operate safely and efficiently for the appropriate level of service. Locations of adjacent interchanges must be evaluated to meet the appropriate weaving and signing criteria.

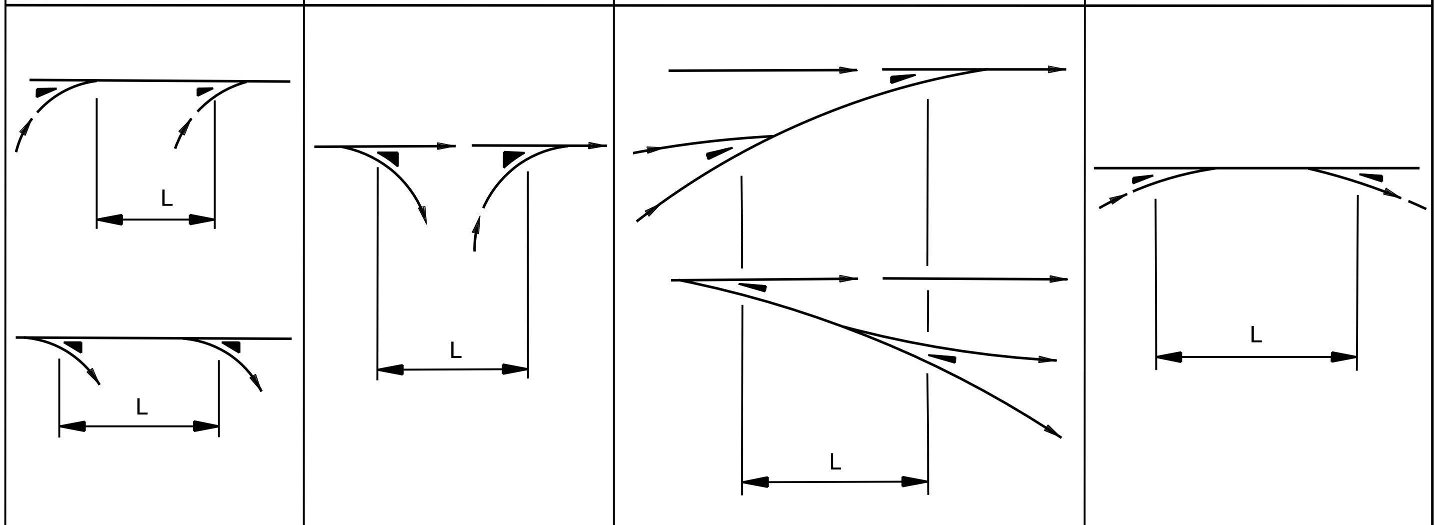
The desirable values usually allow adequate distances for an entering driver to adjust to the freeway environment, for proper weaving maneuvers between entrance and exit ramps and for adequate signing. However, considering the effects of existing streets and highways, traffic operations and environmental considerations, the spacing between adjacent interchanges may vary considerably. Minimum spacing should be 1 mile in urban areas and 3 miles in rural areas, based on crossroad to crossroad spacing. In urban areas, a spacing of less than 1 mile may be developed by using grade separated ramps and collector-distributor roads.

10.3.2 Distance Between Successive Freeway/Ramp Junctions

Successive freeway/ramp junctions may be placed relatively close to each other, especially in urban areas. The distance between the terminals should provide for vehicular maneuvering, signing and capacity. Figure 10.3-A provides recommended guidelines for spacing distances of various freeway/ramp junctions. The criteria in Figure 10.3-A should be considered for the initial planning stages of interchange location. The final decision on the spacing between freeway/ramp junctions should be based on the traffic analysis. Where the distance between the tapers of successive entrance and exit terminals is less than 1500 feet, consider connecting the two terminals with an auxiliary lane and provide a recovery area beyond the exit terminal.

10.3.3 Basic Number of Lanes

The basic number of lanes is the minimum number of lanes designated and maintained over a significant length of a route based on the overall operational needs and traffic volumes of that section. The number of lanes should remain constant over short distances. For example, do not drop a lane at the exit of a diamond interchange and then add it at the downstream entrance simply because the through traffic volume decreases between the exit and entrance ramps. Likewise, do not drop a basic lane between closely spaced interchanges simply because the estimated traffic volume does not warrant the higher number of lanes. Lane drops should only occur where there is an overall reduction in the traffic volumes on the freeway route as a whole.



EN-EN or EX-EX		EX-EN		Directional Ramps		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 Fwy.	CDR or FDR	Full Fwy.	CDR or FDR
Minimum Lengths (L) Measured Between Successive Ramp Terminals									
1000 ft	800 ft	500 ft	400 ft	800 ft	600 ft	2000 ft	1600 ft	1600 ft	1000 ft

FDR - Freeway Distributor Road   CDR - Collector-Distributor Road   EN - Entrance   EX - Exit

Note: The lengths are based on operational experience and the need for flexibility and adequate signing. They should be checked according to the procedure in the Highway Capacity Manual. The larger of the values is suggested for use. Also, a procedure for measuring the length of the weaving section is given in the Highway Capacity Manual.

RAMP TERMINAL SPACING GUIDELINES
Figure 10.3-A

10.3.4 Lane Balance

Lane balance is normally a major concern on high-volume urban freeways and a necessary element to realize efficient traffic operation through an interchange or series of interchanges. After the basic number of lanes is determined, the balance in the number of lanes should be checked on the basis of the following principles:

1. Exits. The number of approach lanes to the highway exit should equal the sum of the number of mainline lanes beyond the exit plus the number of exiting lanes minus one; see Figure 10.3-B. An exception to this principle would be at cloverleaf loop ramp exits that follow a loop ramp entrance or at exits between closely spaced interchanges (e.g., interchanges where the distance between the taper end of the entrance terminal and the beginning taper of the exit terminal is less than 1500 feet and a continuous auxiliary lane is used between the terminals). In these cases, the auxiliary lane may be dropped at a single-lane exit with the number of lanes on the approach roadway being equal to the number of through lanes beyond the exit plus the lane on the exit.

2. Entrances. At entrances, the number of lanes beyond the merging of the two traffic streams should be not less than the sum of the approaching lanes minus one; see Figure 10.3-B.

3. Lane Drops. Reduce the number of travel lanes on the freeway only one lane at a time.

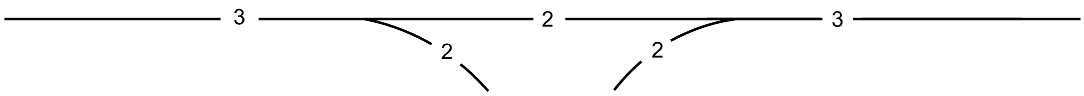
At exits, for example, dropping two mainline lanes at a two-lane exit ramp would violate the principle of lane balance. One lane should provide the option of remaining on the freeway. Lane balance would also prohibit immediately merging both lanes of a two-lane entrance ramp into a highway mainline without the addition of at least one additional lane beyond the entrance ramp. Figure 10.3-B illustrates how to coordinate lane balance and the basic number of lanes at an interchange. Figure 10.3-B also illustrates how to achieve lane balance at the merging and diverging points of branch connections.

10.3.5 Capacity and Level of Service

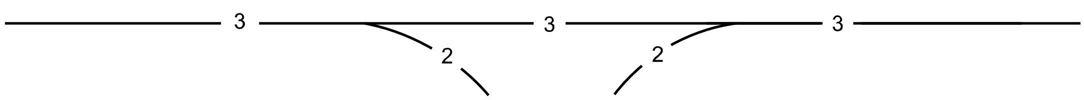
The capacity of an interchange will depend upon the operation of its individual elements which include:

- basic freeway section where interchanges are not present,
- freeway/ramp junctions,
- weaving areas,
- ramp proper,
- collector-distributor roadways, and
- ramp/crossroad intersections.

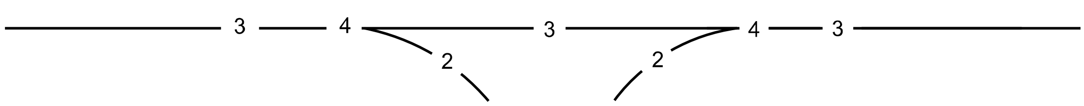
The basic capacity reference is the *Highway Capacity Manual* (HCM). The HCM and applicable software provide the analytical tools required to analyze the level of service for each element listed above. The design year for the interchange and crossroad will typically be the same as that for the freeway (i.e., 20 years).



(a) Lane Balance but no Compliance with Basic Number of Lanes

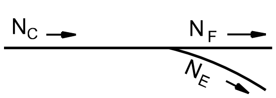


(b) No Lane Balance but Compliance with Basic Number of Lanes



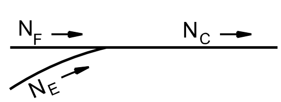
(c) Compliance with Both Lane Balance and Basic Number of Lanes

Exit



$N_C = N_F + N_E - 1$

Entrance



Max: $N_C = N_F + N_E$

Min: $N_C = N_F + N_E - 1$

Where:

N_C = Number of Lanes for Combined Traffic

N_F = Number of Lanes on Freeway

N_E = Number of Lanes on Exit or Entrance Ramp

(d) Lane Balance Equations

COORDINATION OF LANE BALANCE AND BASIC NUMBER OF LANES
Figure 10.3-B

Level of service values presented in Chapter 17 “Freeways” will also apply to interchanges. The level of service of each interchange element should be equal to the level of service provided on the basic freeway section. Individual elements should not operate at more than one level of service below that of the basic freeway section. In addition, the operation of the ramp/crossroad intersection in urban areas should not impair the operation of the mainline. This will likely involve a consideration of the operational characteristics on the minor road for some distance in either direction from the interchange.

10.3.6 Auxiliary Lanes

As applied to interchange design, auxiliary lanes are most often used to comply with the principle of lane balance, to increase capacity, to accommodate weaving or to accommodate entering and exiting vehicles. The operational efficiency of the freeway may be improved if a continuous auxiliary lane is provided between entrance and exit terminals where interchanges are closely spaced. An auxiliary lane may be dropped at an exit if properly signed and designed. The following statements apply to the use of an auxiliary lane within or between interchanges:

1. Within Interchange. Figure 10.3-C provides the basic schematics of alternative designs for adding and dropping auxiliary lanes within interchanges. The selected design will depend upon traffic volumes for the exiting, entering and through movements.

2. Between Interchanges. Where interchanges are closely spaced, the designer should provide an auxiliary lane where the distance between the end of the entrance terminal and beginning of the exit taper is less than 1500 feet.

Design details for exit and entrance ramps are provided in Section 10.4.1 and Section 10.4.2. The design details for freeway lane drops are provided in Section 17.5.1.

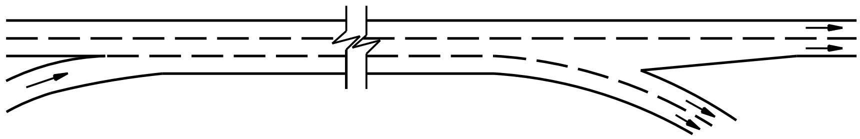
10.3.7 Weaving Sections

Weaving sections are highway segments where the pattern of traffic entering and exiting at contiguous points of access results in vehicular paths crossing each other. The turbulent effect of weaving operations can result in reduced operating speeds and levels of service for the through traffic. Weaving sections may be eliminated at an interchange between two major highways by using direct or semi-direct connections or by using collector-distributor roadways.

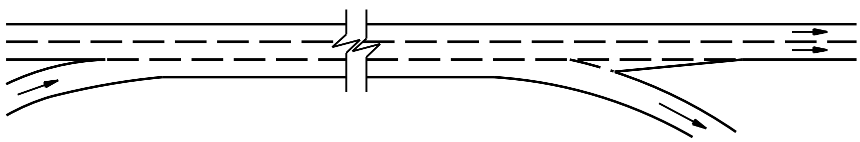
Consider the following for weaving sections:

1. Weave Length. Weaving sections on freeways other than cloverleafs should be at least 1000 feet or the length determined using the *Highway Capacity Manual* (HCM), whichever is greater.

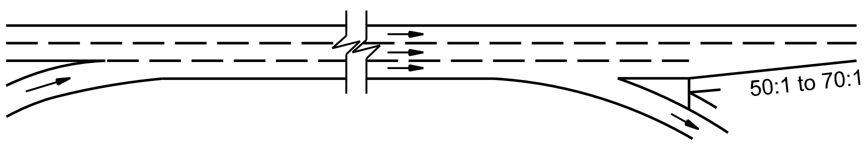
2. Level of Service. The level of service of a weaving section should be the same as that for the adjacent mainline; however, at a minimum, it can be one level lower. A higher volume in weaving sections may be accommodated and their adverse impact on through traffic minimized by providing the weaving section on collector-distributor roadways. Section 10.3.8 discusses the use and design of collector-distributor roadways.



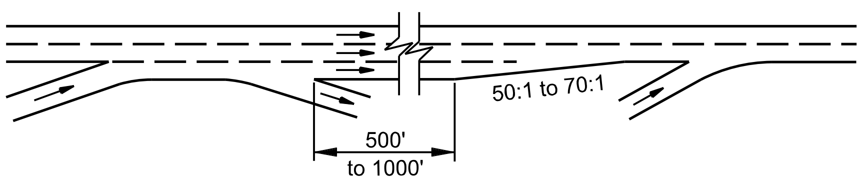
(a) Auxiliary Lane Dropped On Exit Ramp



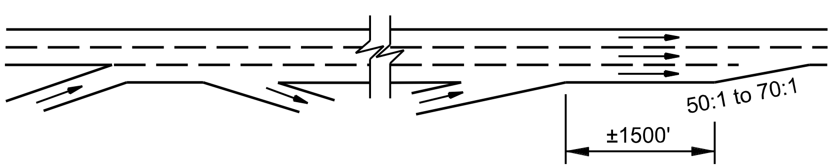
(b) Auxiliary Lane Between Cloverleaf Loops Or Closely Spaced Interchanges Dropped On Single Lane



(c) Auxiliary Lane Dropped At Physical Nose



(d) Auxiliary Lane Dropped Within An Interchange



(e) Auxiliary Lane Dropped Beyond An Interchange

AUXILIARY LANES WITHIN AN INTERCHANGE
Figure 10.3-C

10.3.8 Collector-Distributor Roadways

10.3.8.1 Usage

A collector-distributor (C-D) roadway is an auxiliary roadway parallel to and separated from the main traveled way that serves to collect and distribute traffic from multiple access points. It provides greater capacity and permits higher operating speeds to be maintained on the main traveled way. C-D roadways may be provided at single interchanges, through two adjacent interchanges or, in urban areas, continuously through several interchanges. Figure 10.2-H illustrates a schematic of a C-D roadway within a full cloverleaf interchange.

Usually, interchanges designed with single exits are superior to those with two exits, especially if one exit 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 the operational efficiency of the entire interchange. C-D roadways use the single exit approach to improve the interchange operational characteristics. C-D roadways will:

- remove weaving maneuvers from the mainline and transfer them to the slower speed C-D roadways,
- provide high-speed single exits and entrances from and onto the mainline,
- satisfy driver expectancy by placing the exit before the grade separation structure,
- simplify signing and the driver decision-making process, and
- provide uniformity of exit patterns.

C-D roadways are most often warranted when traffic volumes (especially in weaving sections) are so high that the interchange cannot operate at an acceptable level of service. They also may be warranted where the speed differential between weaving and non-weaving vehicles is significant.

10.3.8.2 Design

When designing C-D roadways, consider the following:

1. Design Speed. The design speed of a C-D roadway usually ranges from 40 to 65 miles per hour. Typically, use a design speed within 10 miles per hour of the mainline design speed.

2. Lane Balance. Maintain lane balance at the exit and entrance points of the C-D roadways; see Section 10.3.4.

3. Width. C-D roadways may be one or two lanes, depending upon the traffic volumes and weaving conditions. C-D roadways are typically designed similar to ramps with traveled way widths of either 16 feet (1 lane) or 24 feet (2 lanes).

4. Separations. The separation between the C-D roadway and mainline should be as wide as practical. At a minimum, the separation should allow for shoulder widths equal to that on the mainline and for a suitable barrier to prevent indiscriminate crossovers.

5. Terminal Designs. Section 10.4 discusses the design of freeway/ramp junctions. These criteria also apply to C-D roadway/ramp junctions.

10.3.9 Route Continuity

The major route should flow continuously through an interchange. For freeway and expressway routes that change direction, the driver should not be required to change lanes or exit to remain on the major route. Route continuity without a change in the basic number of lanes is consistent with driver expectancy, simplifies signing and reduces the decision demands on the driver. Interchange configurations should not necessarily favor the heavier traffic movement. There may occasionally be sites where it is advisable to design the interchange to provide route continuity despite the traffic volume movements.

10.3.10 Uniformity

Interchange configurations along a route should be uniform from one interchange to another. All ramps should exit and enter on the right except under highly unusual conditions. Dissimilar arrangements between interchanges can cause confusion resulting in undesirable lane switches, reduced speeds, etc., especially in urban areas where interchanges are closely spaced.

10.3.11 Signing and Marking

Proper interchange operations depend on the compatibility between its geometric design and the traffic control devices at the interchange. The proper application of signs and pavement markings will increase the clarity of paths to be followed, safety and operational efficiency. The logistics of signing along a highway segment will also influence the minimum acceptable spacing between adjacent interchanges. The *MUTCD* provides guidelines and criteria for the placement of traffic control devices at interchanges.

10.3.12 Ramp Metering

Ramp metering may be used to improve freeway operations. Ramp metering consists of signals installed on entrance ramps before the entrance terminal to control the number of vehicles entering the freeway. The traffic designer will determine the need for ramp metering. If used, the roadway designer will need to coordinate with the traffic designer to determine the placement of the ramp signal to ensure that there is a sufficient storage area before the ramp signal and that sufficient acceleration distance is available beyond the signal to allow a vehicle to reach freeway speed.

10.3.13 Grading and Landscaping

Consider the grading around an interchange early in the design process. Alignment, fill-and-cut sections, median widths, lane widths, drainage, structural design and infield contour grading all affect the aesthetics of the interchange. Properly graded interchanges allow the overpassing structure to blend naturally into the terrain. In addition, ensure that the crossroad and ramp slopes are not so steep that they compromise safety and can support plantings to prevent erosion and enhance the appearance of the area. Flatter slopes also allow easier maintenance. Transitional grading between cut-and-fill slopes should be long and natural in appearance. The designer must ensure that plantings will not affect the sight distance within the interchange and that larger plantings are a significant distance from the traveled way.

Include a contour grade detail for interchanges in the plans.

10.3.14 Operational/Safety Considerations

Operations and safety are important considerations in interchange design. The following summarizes several general factors:

1. Exit Ramps. For exit ramps, consider the following:

    a. Signing. Proper advance signing of exits is essential to allow necessary lane changes before the exit.

    b. Deceleration. Provide sufficient distance to allow safe deceleration from the freeway design speed to the design speed of the first governing geometric feature on the ramp, typically a horizontal curve.

2. Entrance Ramps. Provide an acceleration distance of sufficient length to allow a vehicle to attain an appropriate speed for merging. Where entrance ramps enter the mainline on an upgrade, the acceleration distance may need to be lengthened, or an auxiliary lane may be required to allow vehicles to reach a safe speed prior to merging.

3. Driver Expectancy. Ensure that the interchange is designed to conform to the principles of driver expectancy. These may include the following:

    a. Left-Hand Exits and Entrances. Avoid left-hand exit or entrance terminals. Drivers expect single-lane exit and entrance terminals to be located on the right side of the freeway.

    b. Horizontal Alignment. Do not locate exit ramps so that it gives the appearance of a continuing mainline tangent as the mainline curves to the left.

    c. Consistency. Do not mix operational patterns between interchanges, lane continuity or interchange types.

    d. Lane Balance. Provide lane balance and the basic number of lanes on the freeway.

    e. Spacing. Provide sufficient spacing between interchanges to allow proper signing distances to decision points.

4. Roadside Safety. Because of the typical design features at interchanges, many fixed objects may be located within interchanges (e.g., signs at exit gores, bridge piers, rails). Avoid locating these objects near decision points, make them breakaway or shield them with barriers or impact attenuators. See the AASHTO *Roadside Design Guide* for detailed guidance on roadside safety.

5. Traffic-Controlled Ramp Terminals. The designer must ensure that the ramp/crossroad intersection has sufficient capacity so that the queuing traffic at the crossroad intersection does not backup onto the freeway. Also, sufficient access control and intersection sight distance must be maintained along the crossroad to allow the ramp intersection to work properly. Provide sufficient sight distance to the ramp/crossroad traffic control devices.

6. Wrong-Way Maneuvers. Provide channelized medians, islands and/or adequate signing to minimize wrong-way possibilities. Avoid designs that may result in poor visibility, confusing ramp arrangements or inadequate signing.

7. Pedestrians and Bicyclists. Use signing and lane markings to increase awareness of pedestrians and bicyclists. Signing, crosswalks, barriers, over and underpasses, bridge sidewalks and other traffic control devices may be required to manage traffic movements and to control pedestrian and bicycle movements.

10.3.15 Geometric Design Criteria

Design all roadways through an interchange with the same criteria as used for the approaches including design speed, sight distance, horizontal and vertical alignment, cross section and roadside safety elements. In addition, consider the following:

1. Design Year. Typically, use a 20-year design period based on the anticipated project letting date.

2. Design Speed (Crossroad). The crossroad design speed should be based on the functional classification and urban or rural classification; see the geometric design tables in Chapters 14 “Local Roads and Streets,” Chapter 15 “Collector Roads and Streets” and Chapter 16 “Rural and Urban Arterials.”

3. Horizontal Alignment. In general, design the alignment of the freeway and crossroad through the interchange on a tangent. Where this is not practical, consider the following:

    a. Freeway Mainline. Minimize locating exit terminals where the freeway mainline curves to the left. If it cannot be avoided, provide an abrupt exit taper.

    b. Freeway/Ramp Junctions. Design the freeway alignment so that only one exit terminal departs from the mainline curving to the right, or design the mainline curve to lie entirely within the limits of the interchange and away from the exit and entrance terminals.

    c. Superelevation. Desirably, design the horizontal alignment so that superelevation and superelevation transitions will not be required through the freeway/ramp junctions or through the ramp/crossroad intersection.

    d. Crossroad. Where a curve is necessary, provide a significantly large horizontal curve so that superelevation is not required on the crossroad, if practical.

4. Vertical Alignment. Vertical profiles for both roadways through the interchange should be as flat as practical. Where compromises are necessary, use the flatter grade on the major facility. In addition, the designer should consider the following:

a. Sight Distance. To improve the sight distance to exit gores, locate exit ramp terminals and major divergences where the mainline is on an upgrade.

b. Ramps. Ramps should depart from the mainline where there will be no vertical curvature to restrict visibility along the ramp. Avoid ramp designs that drop out of sight. Also, provide flat approach grades adjacent to the crossroad. For additional information on storage platforms at the ramp/crossroad intersection, see Section 9.2.7.

c. Exit Ramp Terminals. Where a freeway is proposed to cross over the crossroad, locate the exit ramp terminals on the mainline no closer than 1000 feet from the high point of a crest vertical curve on the mainline. This will ensure that no hidden ramps exist and will provide for safer operations at the exit ramp terminal.

d. Turning Trucks. Large trucks may become unstable when executing a nonstop, left turn from a crossroad on a downgrade. The combination of a downgrade, sharp turning maneuvers onto a ramp and reverse superelevation may produce instability in large trucks. Therefore, the maximum grade for all crossroads associated with these conditions is desirably 2 percent through the ramp/crossroad terminal. For existing crossroads to remain in place, limit the downgrade to 3 percent. At a maximum, limit the up and downgrades to 4 percent.

5. Sight Distance. Because of the additional demand placed on the driver at an interchange, the designer should consider the following sight distance elements:

a. Stopping Sight Distance. Provide adequate stopping sight distance on both intersecting highways throughout the interchange and on all ramps. Check both the vertical and horizontal alignment to ensure that the location of piers, abutments, structures, bridge rails, vertical curves, etc., will not restrict sight distance. Chapter 5 “Horizontal Alignment” discusses the application of horizontal sight distance. Chapter 6 “Vertical Alignment” discusses the application of vertical sight distance.

b. Decision Sight Distance. Desirably, provide decision sight distance to all decision points (e.g., exit and entrance terminals); see Section 4.3. Desirably, at exit ramps, use the pavement surface for the height of object (i.e., 0.0 inches). Driver expectancy should not be violated.

c. Intersection Sight Distance. Section 4.4 discusses intersection sight distance (ISD), which is also applicable at ramp/crossroad intersections (non-merging sites). Section 10.6 provides additional ISD guidance that should be considered at ramp/crossroad intersections that are stop controlled.

6. Ramp/Crossroad Intersections. When designing the ramp/crossroad intersection, consider the following:

a. Angle of Ramp Intersection. To determine the appropriate angle for the ramp/crossroad intersection, see Section 9.2.6.

b. Access Control. To determine the required length of access control along the crossroad at the interchange, see Section 3.8.

c. Left-Turn Lanes. Select the appropriate left-turn lane lengths based on the design speed of the crossroad and/or the required storage lengths; see Section 9.5.

d. Turning Movements. Check the ramp/crossroad intersection with the applicable design vehicle turning template or use a computer-simulated turning template program. Use the WB-62 design vehicle for determining turning radii, curb locations, median noses, etc., at the ramp/ crossroad intersection. Use the WB-67 design vehicle for determining storage lengths (e.g., left-turn lanes), median widths, etc., at the ramp/crossroad intersection.

7. Mainline/Crossroad Point of Intersection. Once Items 1 through 6 above have been determined, the designer must decide where the mainline alignment best intersects with the crossroad. The overall size of the interchange, crossroad grade lines, required length of access control along the crossroad, access to property at the ends of access control on the crossroad, and topography are the most influential factors in this determination. Complete this investigation before the detailed design of an interchange is initiated.

10.3.16 Reviewing for Ease of Operation

The designer should review the proposed design from the driver’s perspective. This involves tracing all possible movements that an unfamiliar motorist would drive through the interchange. Review the plans for areas of possible confusion, proper signing, ease of operation and to determine if sufficient weaving distance and sight distances are available. Review both day and nighttime operations. Consider the peak-hour volumes, number of traffic lanes, etc., to determine the type of traffic the driver may encounter.

10.4 FREEWAY/RAMP JUNCTIONS

10.4.1 Exit Ramps

10.4.1.1 Types of Exit Ramps

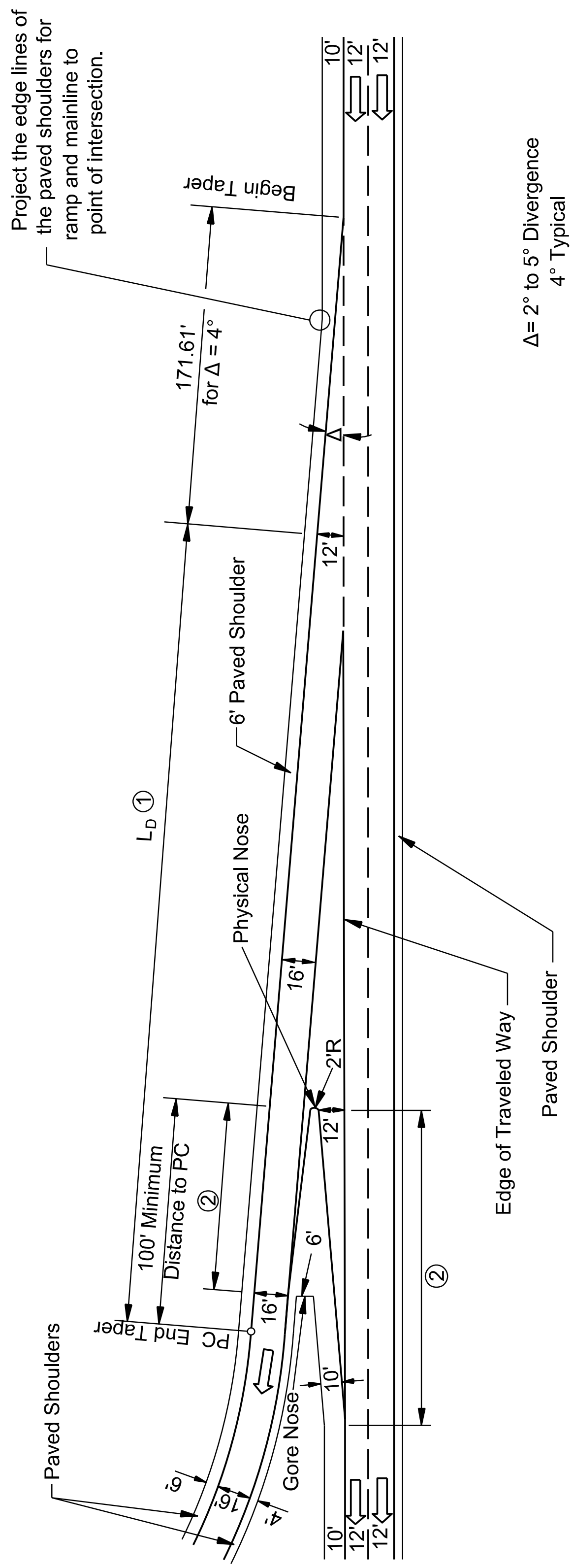
There are two basic types of exit freeway/ramp junctions — taper design and parallel design. Figures 10.4-A, 10.4-B and 10.4-C illustrate these designs. For most new and reconstructed ramps, SCDOT prefers to use the taper design. However, the designer may consider using the parallel design where:

- a ramp exit is just beyond an overpass structure and there is insufficient sight distance available to the ramp gore;
- the need is satisfied for a continuous auxiliary lane (see Section 10.3.6);
- the geometrics of the exit ramp are such that the taper design cannot satisfy the length needed for deceleration; or
- the exit ramp departs from a horizontal curve on the mainline. In this case, the parallel design is less confusing to through traffic and will normally result in smoother operations. It is also easier to design the superelevation transition with a parallel design.

10.4.1.2 Taper Rates

The taper rate applies to the rate at which the ramp diverges from the mainline. The following taper rates apply:

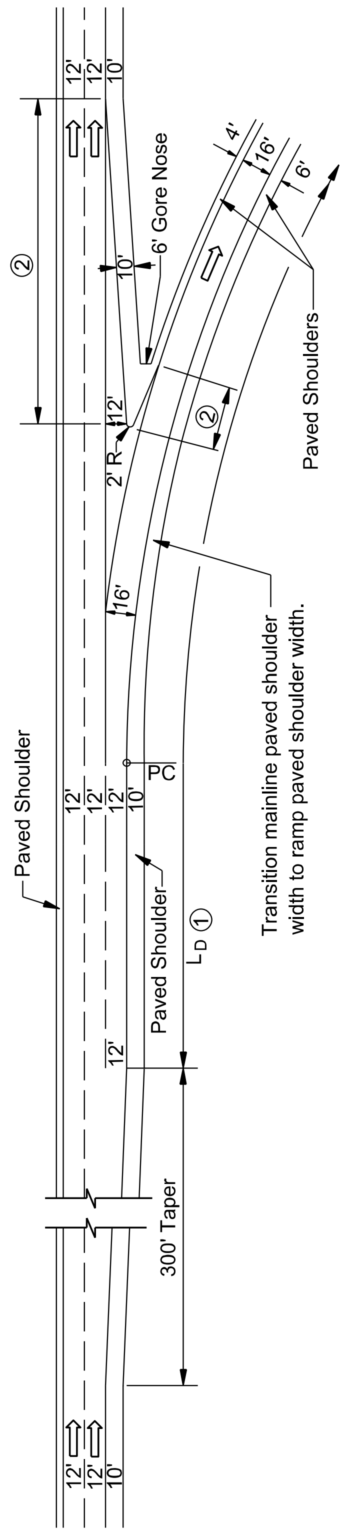
1. Taper Exit Design. The taper angle can vary between 2 and 5 degrees. For the typical SCDOT ramp design, the divergence angle is 4 degrees as illustrated in Figure 10.4-A.
2. Parallel Exit Design. The taper rate applies to the beginning of the parallel lane. This distance is typically 300 feet (i.e., 25:1) as illustrated in Figures 10.4-B and 10.4-C.



Notes: 1. L_D is the deceleration distance required for a vehicle to slow down from the mainline design speed to the design speed of the first geometric control on the ramp; see Section 10.4.1.3.

2. See Figure 10.4-G for the taper rate (z) to transition the offset physical nose to the normal traveled way width.

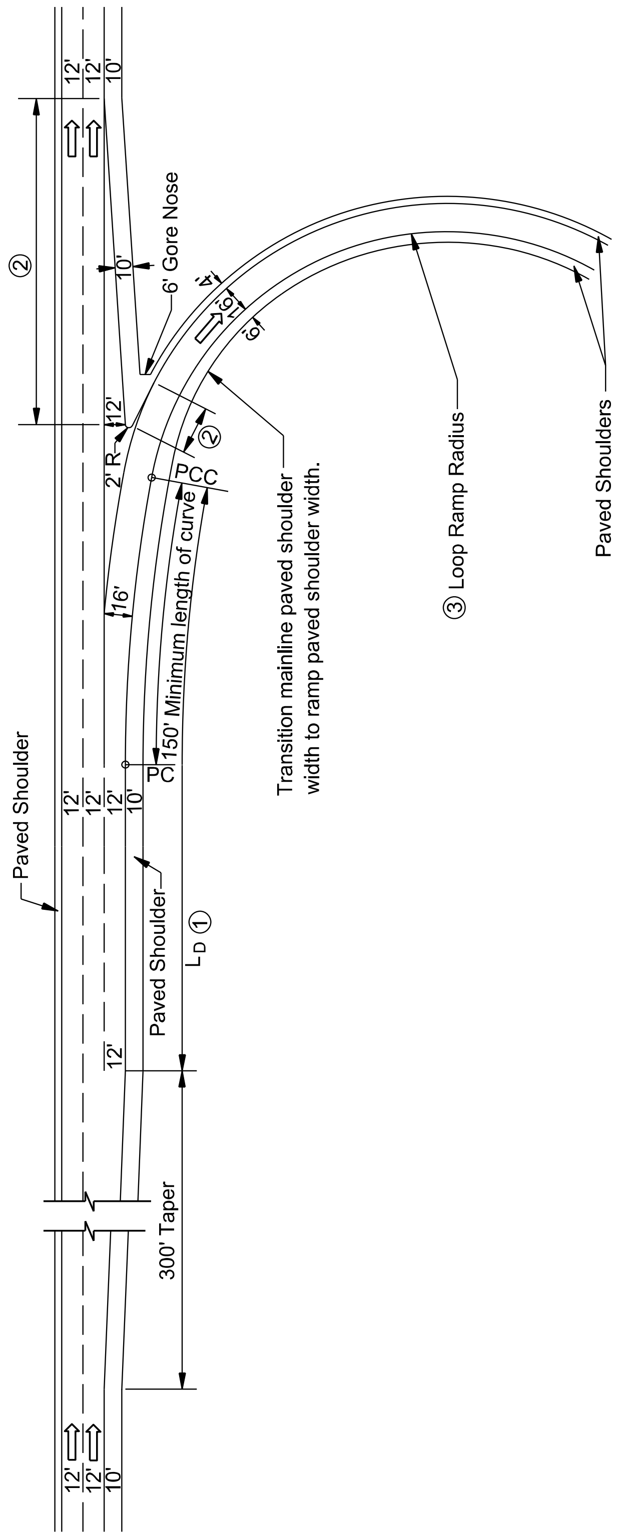
TAPER EXIT RAMP
Figure 10.4-A



Notes: 1. L_D is the deceleration distance required for a vehicle to slow down from the mainline design speed to the first geometric control on the ramp. See Section 10.4.1.3.

2. See Figure 10.4-G for the taper rate (z) to transition the offset physical nose to the normal traveled way width.

PARALLEL EXIT RAMP
Figure 10.4-B



Notes: 1. L_D is the deceleration distance required for a vehicle to slow down from the mainline design speed to the first geometric control on the ramp. See Section 10.4.1.3.

2. See Figure 10.4-G for the taper rate (z) to transition the offset physical nose to the normal traveled way width.

3. Offset Loop Ramp Radius 24 feet from outside edge off mainline travel way.

PARALLEL EXIT FOR LOOP RAMP
Figure 10.4-C

10.4.1.3 Deceleration

Sufficient deceleration is needed to safely and comfortably allow an exiting vehicle to depart from the mainline. The following will apply:

1. Taper Exit. All deceleration should occur within the full width of the deceleration lane. The length of deceleration will depend upon the design speed of the mainline and design speed of the first governing geometric control on the ramp, typically a horizontal curve. This distance is measured from where the ramp becomes 12 feet wide to the first geometric control.

2. Parallel-Lane Exit. Design the departure curve or taper based on the design speed of the roadway being departed. The deceleration length begins where the full width of the parallel lane becomes available and ends where the departure curve of the ramp begins; see Figures 10.4-B and 10.4-C.

Figure 10.4-D provides the deceleration distances for various combinations of highway design speeds and ramp design speeds. If the deceleration distance is on a downgrade of 3 percent or more, adjust the deceleration distance according to the criteria in Figure 10.4-E.

* * * * * * * * *

Example 10.4-1

Given:

Given:	Highway Design Speed	=	70 miles per hour
	First Exit Curve Design Speed	=	45 miles per hour
	Average Grade	=	5 percent downgrade

Problem: Determine length of deceleration required.

Solution: Figure 10.4-D yields a minimum deceleration length of 390 feet on the level. According to Figure 10.4-E, this should be increased by 1.35.

Therefore: $L_D = (390)(1.35)$
$L_D = 527$ feet

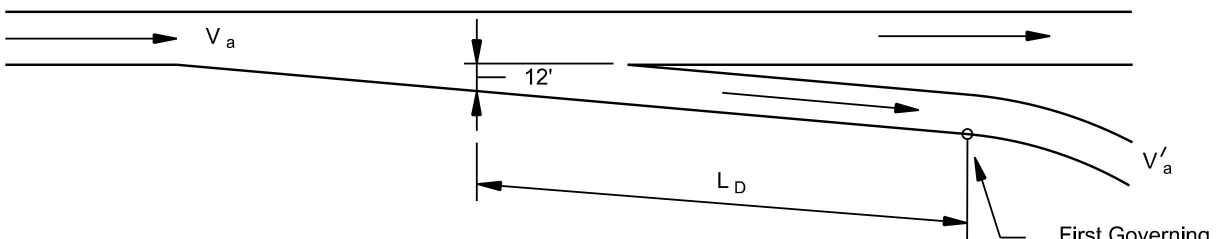
Provide a 530-foot deceleration length from the full width of the exit lane to the PC of the first exit curve.

* * * * * * * * *

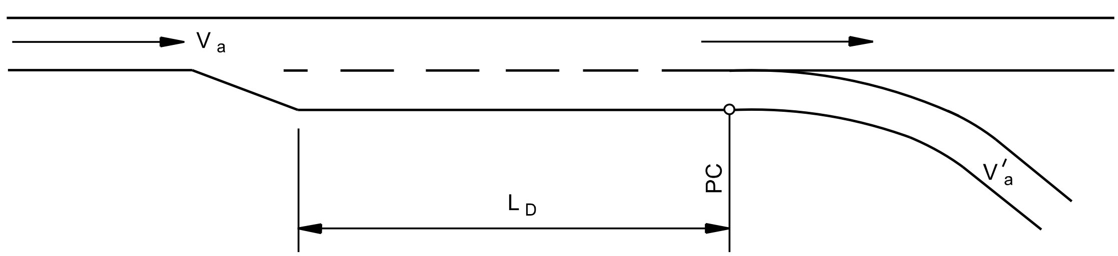
10.4.1.4 Sight Distance

Desirably, provide decision sight distance approaching a freeway exit. At a minimum, this sight distance should exceed the stopping sight distance by 25 percent. This sight distance should be available throughout the freeway/ramp junction (e.g., from the beginning taper to the gore nose; see Figures 10.4-A and 10.4-B). This sight distance is particularly important for exit loops immediately beyond a structure. Vertical curvature or bridge piers can obstruct the exit points if not carefully designed. The desirable height of object will be 0.0 feet (the roadway surface); however, it is acceptable to use 2 feet.

Design Speed of Highway (mph)	Speed Reached (mph)	L_D = Length of Deceleration (ft)								
		For Design Speed of First Governing Geometric Control (mph)								
		Stop	15	20	25	30	35	40	45	50
		For Average Running Speed (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



Taper Design



Parallel Design

Notes:

1. *The deceleration lengths are calculated from the distance needed for a passenger car to decelerate from the average running speed of the highway mainline to the average running speed of the first governing geometric control.*
2. *These values are for grades less than 3 percent. See Figure 10.4-E for steeper downgrades.*
3. *Select the actual design speed of the mainline and ramp when using this figure.*

LENGTH FOR DECELERATION
Figure 10.4-D

Direction of Grade	Ratio of Deceleration Length on Grade (G) to Length on Level		
	G < 3%	3% ≤ G < 5%	5% ≤ G < 6%
Downgrade	1.0	1.2	1.35
Upgrade	1.0	0.9	0.8

Notes: 1. Figure applies to all highway design speeds.

2. The grade in the table is the average grade over the distance used for measuring the length of deceleration. See Figures 10.4-A and 10.4-B.

GRADE ADJUSTMENTS FOR DECELERATION LENGTHS
Figure 10.4-E

10.4.1.5 Horizontal Alignment

Develop the superelevation for horizontal curves at the freeway/ramp junction based on the principles of superelevation as discussed in Section 5.3 for mainline highways. In addition, the following criteria are applicable to superelevation development at freeway ramp/junction exits:

1. Design Speed. Desirably, the design speed of the mainline roadway will be used as the design speed for any horizontal curves at the freeway/ramp exit. As discussed in Section 10.4.1.3, the freeway/ramp exit should provide sufficient distance for a vehicle to decelerate from the mainline design speed to the design speed of the first controlling design element of the exit ramp. This could be a horizontal curve in the vicinity of the exit gore. If the necessary deceleration distance is available, the design speed of the horizontal curve at a minimum may be equal to the design speed of the ramp proper; see Section 10.5.

2. Maximum Superelevation. The e_{max} that is applicable to the mainline (see Section 5.3) will also apply to horizontal curves at the freeway/ramp exit. In most cases, this will be e_{max} = 8.0 percent.

3. Minimum Curve Radius. Use the applicable e_{max} figure in Section 5.3 to determine the minimum radius for horizontal curves at freeway/ramp exits. The designer will use the selected design speed and appropriate design superelevation (e_d) to determine the minimum radius.

4. Transition Length. The designer must transition the exit ramp cross slope on tangent (typically 2.00 percent) to the superelevation rate for the horizontal curve. The following applies:

    a. The transition should not begin until the exit ramp has reached a minimum 12-foot width.

    b. The maximum relative gradient should not exceed the criteria in Figure 5.3-A. The relative gradient is measured between the outside edge of ramp traveled way and the inside edge of ramp traveled way.

c. If practical, approximately 67 percent of the transition length should be on the tangent and approximately 33 percent on the curve.

5. Point of Revolution. The designer may choose to put the point of revolution on either the inside or outside of the ramp traveled way.

10.4.1.6 Cross Slope Rollover

The cross slope rollover is the algebraic difference between the transverse slope of the through lane and the transverse slope of the exit lane and/or gore. The following will apply:

1. Up to Physical Nose. The cross slope rollover should not exceed 5 percent.

2. From Physical Nose to Gore Nose. The cross slope rollover should not exceed 7 percent.

3. Drainage Inlets. Where required, these are normally placed between the physical gore and gore nose. The presence of drainage inlets may require two breaks in the gore cross slope. These breaks should meet the criteria in Items 1 or 2 above, depending on the inlet location.

See Section 10.4.1.8 for gore nose definitions.

10.4.1.7 Shoulders

The wider right shoulder of the mainline must be transitioned to the narrower shoulder of the ramp (i.e., 10 foot paved to 6 foot paved). The shoulder width should be transitioned as shown in Figures 10.4-A and 10.4-B.

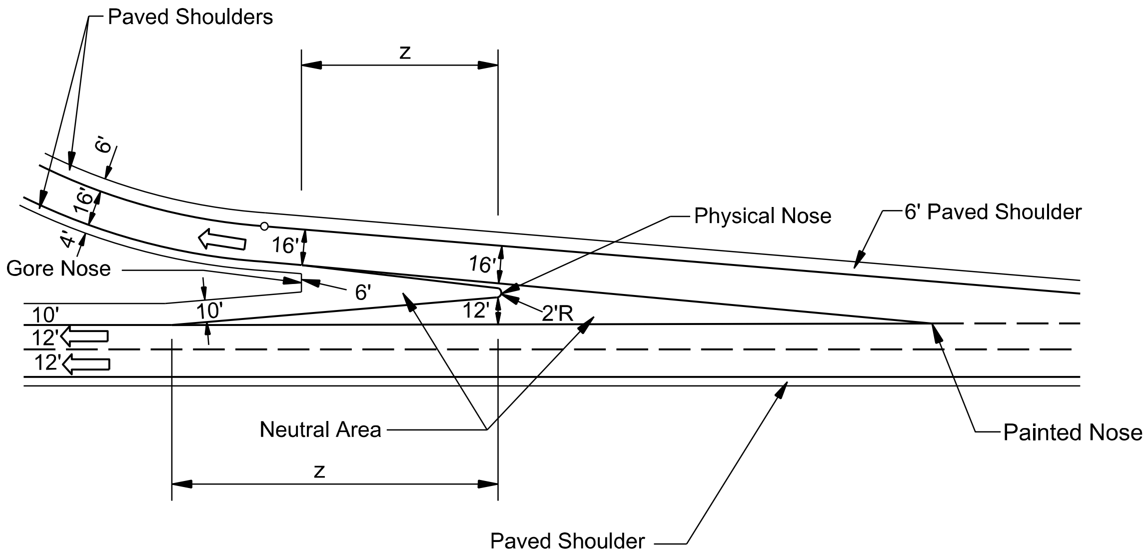
10.4.1.8 Gore Area

The gore area is normally considered both the paved triangular area between the through lane and the exit ramp, plus the graded area that may extend a few hundred feet downstream beyond the gore nose. The following definitions will apply (see Figure 10.4-F):

1. Painted Nose. This is the point (without width) where the pavement striping on the left side of the ramp converges with the stripe on the right side of the mainline traveled way.

2. Physical Nose. This is the point where the ramp and mainline shoulders converge. As illustrated in Figure 10.4-F, the physical nose is rounded with a 2-foot radius.

3. Gore Nose. This is the point where the paved shoulder ends and the grassed area begins as the ramp and mainline diverge from one another, as illustrated in Figure 10.4-F.



GORE AREA CHARACTERISTICS
Figure 10.4-F

Consider following when designing the gore:

1. Obstacles. If practical, the area beyond the gore nose should desirably be free of all obstacles (except the ramp exit sign) for at least 100 feet beyond the gore nose. Any obstacles within approximately 300 feet of the gore nose must be made breakaway or shielded by a barrier or impact attenuator.

2. Transitions. Figure 10.4-G provides the minimum taper rates (z) that should be used to transition the physical nose offset to the normal traveled way width.

3. Side Slopes. The graded area beyond the gore nose should be as flat as practical. If the elevation between the exit ramp or loop and the mainline increases rapidly, this may not be practical. These areas will likely be non-traversable, and the gore design must shield the motorist from these areas. At some sites, the vertical divergence of the ramp and mainline will warrant protection for both roadways beyond the gore; see AASHTO *Roadside Design Guide*.

4. Cross Slopes. The paved triangular gore or neutral area between the through lane and exit ramp should be safely traversable. The cross slope is the same as that of the mainline (typically 2.00 percent) from the painted nose up to the physical nose. Beyond this point, the gore area is depressed with cross slopes of 2 to 4 percent. See Section 10.4.1.6 for criteria on breaks in cross slopes within the gore area.

Design Speed of Approach Highway (mph)	Taper Rate (z) for Offset Nose
40	20:1
45	22.5:1
50	25:1
55	27.5:1
60	30:1
65	32.5:1
70	35:1
75	37.5:1

MINIMUM TAPER RATES BEYOND THE OFFSET PHYSICAL NOSE
Figure 10.4-G

10.4.2 Entrance Ramps

10.4.2.1 Types

There are two basic types of entrance freeway/ramp junctions — taper design and parallel design; see Figures 10.4-H and 10.4-I. For most entrance ramps, the parallel design is preferred considering the following:

1. Level of Service. Where the level of service for the freeway/ramp merge approaches capacity, a parallel design can be lengthened to allow the driver more time and distance to merge into the through traffic.

2. Acceleration Length. Additional acceleration length can be more easily provided by the parallel design than the taper design.

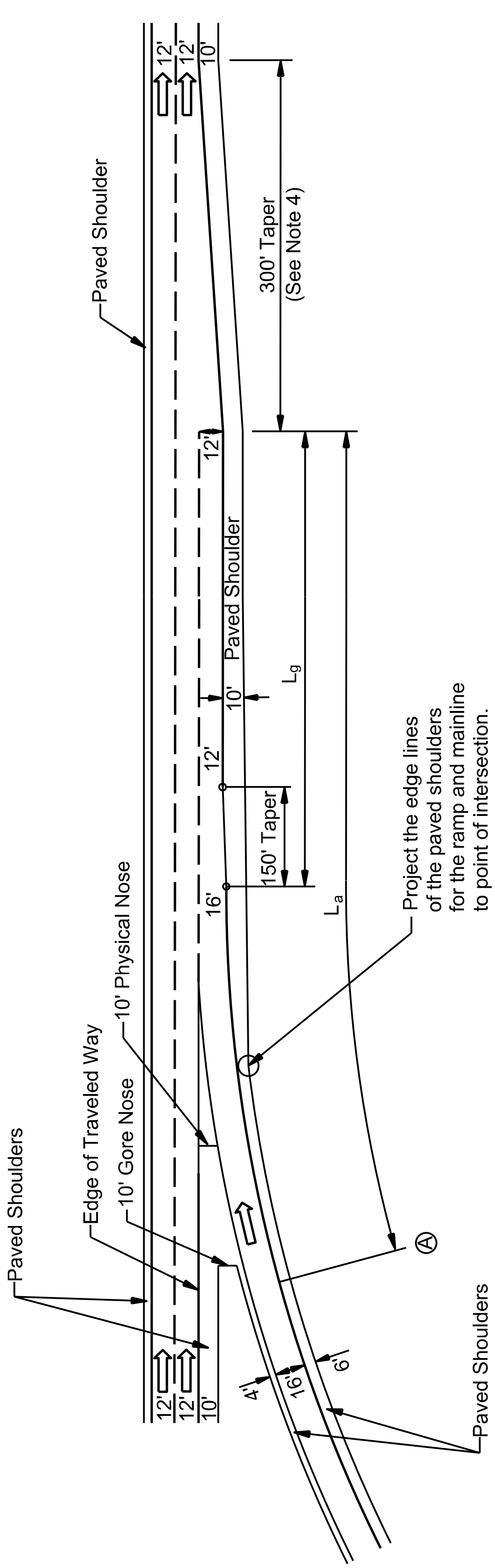
3. Sight Distance. Where there is insufficient sight distance available for the driver to merge into the mainline (e.g., where there are sharp curves on the mainline), the parallel entrance ramp allows a driver to use the side-view and rear-view mirrors to more effectively locate gaps in the mainline traffic.

10.4.2.2 Taper Rates

The following taper rates apply to the entrance design:

1. Parallel Design. For parallel-lane entrance ramps, the taper applies to the merge point at the end of the parallel portion of the ramp. The minimum distance is 300 feet as illustrated in Figure 10.4-H.

2. Taper Design. This rate applies to the rate at which the ramp connects with the mainline. The rate should be between 50:1 (minimum) and 70:1 (desirable) for merges onto a major highway and 25:1 for merges onto a crossroad. See Figure 10.4-I.

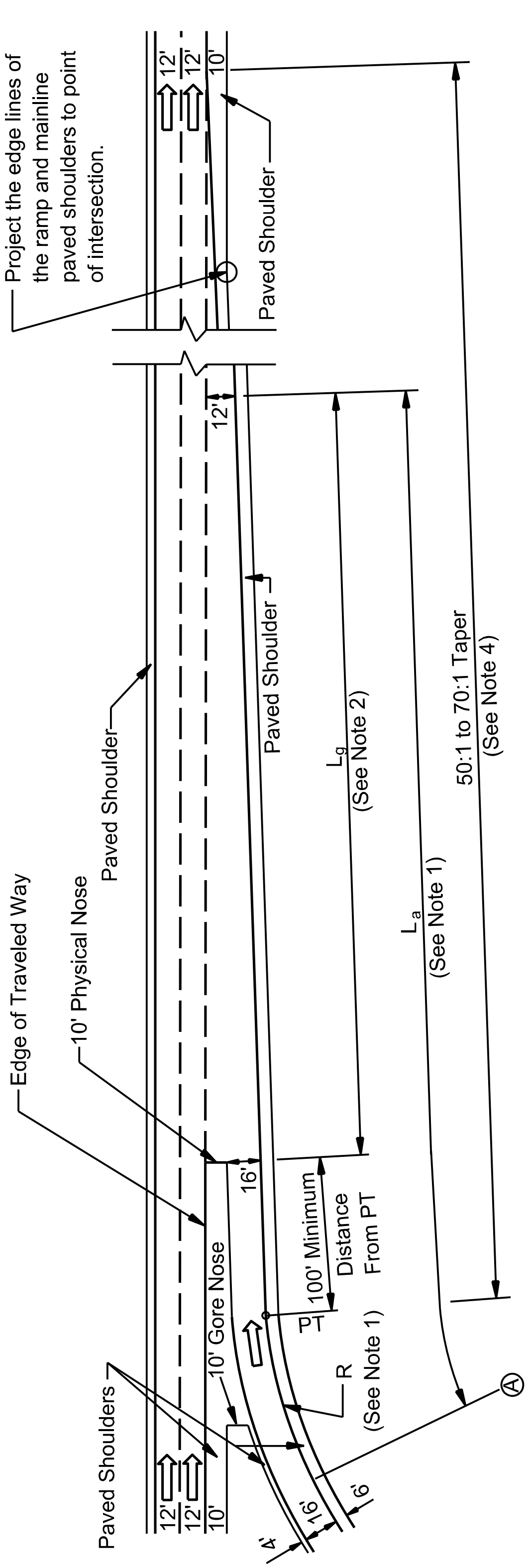


Notes:

1. L_a is the required acceleration length. Point Ⓐ controls the safe speed on the ramp. L_a should not start on the curvature of the ramp unless the ramp radius is ≥ 1000 feet; see Section 10.4.2.3.
2. L_g is the required gap acceptance length. L_g should be a minimum of 300 feet.
3. Use the greater distance of L_a or L_g for determining the ramp entrance length.
4. The taper rate should be 50:1 to 70:1 if L_g is 2500 feet or greater; see Section 17.5.1.

PARALLEL-LANE ENTRANCE RAMP

Figure 10.4-H



Notes: 1. *L_a is the required acceleration length. Point Ⓐ controls the safe speed on the ramp. L_a should not start on the curvature of the ramp unless the ramp radius is ≥ 1000 feet; see Section 10.4.2.3.*

2. *L_g is the required gap acceptance length. L_g should be a minimum of 300 feet to 500 feet from the 10-foot nose width.*
3. *Use the greater distance of L_a or L_g for determining the ramp entrance length.*
4. *The transition taper rate of 50:1 to 70:1 is provided from the PT to the end of the taper.*

TAPER ENTRANCE RAMP
Figure 10.4-I

10.4.2.3 Acceleration

Driver comfort, traffic operations and safety will be improved if sufficient distance is available for acceleration. The length for acceleration will primarily depend upon the design speed of the last controlling horizontal curve on the entrance ramp and the design speed of the mainline. When determining the acceleration length, the designer should consider the following:

1. Passenger Cars. Figure 10.4-J provides the minimum lengths of acceleration for passenger cars. The acceleration distance is measured from the PT of the last controlling horizontal curve to the point at which the acceleration lane becomes less than 12 feet in width; see Figures 10.4-H and 10.4-I. Also, see Item 3 to determine how the horizontal curve interrelates with determining the acceleration distance. Where upgrades meet or exceed 3 percent over the acceleration distance, adjust the acceleration length according to the values presented in Figure 10.4-K.

   The acceleration lengths provide sufficient distance for the acceleration of passenger cars. Where the mainline and ramp will carry traffic volumes approaching the design capacity of the merging area, the available acceleration distance should be at least 1200 feet, exclusive of the taper, to provide additional merging opportunities. This distance is measured from the PT of the ramp entrance curve.

2. Trucks. Where there are a significant number of trucks to govern the design of the ramp, consider providing the truck acceleration distances shown in Figure 10.4-L. Typical areas where trucks might govern the ramp design include weigh stations, rest areas, truck stops and transport staging terminals. At other freeway/ramp entrances, consider truck acceleration distances where there is substantial truck traffic and where:

   - there is a significant crash history involving trucks that can be attributed to an inadequate acceleration length, and/or

   - there is an undesirable amount of vehicular delay at the junction attributable to an inadequate acceleration length.

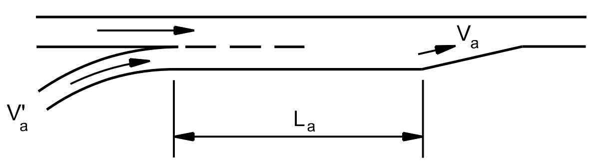
   Where upgrades meet or exceed 3 percent, the truck acceleration distances may be adjusted for grades. Figure 6.4-B provides the performance criteria for trucks on descending and ascending grades. Before providing any additional acceleration length, the designer must consider the impacts of the added length (e.g., additional construction costs, wider structures, right-of-way impacts).

3. Horizontal Curves. In many cases, the speed of a vehicle entering the mainline from the ramp will be dictated by a horizontal curve immediately before the freeway/ramp junction. Determine the design speed of this horizontal curve using the criteria in Section 5.3. Use this speed to read into Figure 10.4-J or Figure 10.4-L to determine the necessary acceleration length.

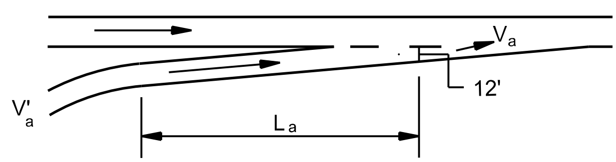
   Two exceptions to the above exist — entrance ramps at diamond interchanges and short entrance ramps. In these cases, determine the acceleration distance by that distance needed to accelerate from zero (at the beginning of the ramp) to the mainline design speed. The designer should check to determine if this distance governs.

In all cases, the curve preceding the freeway/ramp entrance should have a radius of 1000 feet or greater.

Design Speed of Highway (mph)	Speed Reached at End of Full Lane Width (mph) (V_a)	L_a = Length of Acceleration (ft)								
		For Entrance Curve Design Speed (mph)								
		Stop	**15**	**20**	**25**	**30**	**35**	**40**	**45**	**50**
		For Average Running 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



Parallel Type



Taper Type

Notes:

1. *The acceleration lengths are calculated from the distance needed for a passenger car to accelerate from the average running speed of the entrance curve to reach a speed (V_a) of approximately 5 miles per hour below the average running speed on the mainline.*
2. *These values are for grades less than 3 percent. See Figure 10.4-K for adjustments for steeper upgrades.*
3. *Select the actual design speed of the mainline and ramp when using this figure.*

LENGTHS FOR ACCELERATION
(Passenger Cars)
Figure 10.4-J

Design Speed of Highway (mph)	Ratio of Acceleration Length on Grade (G) to Length on Level For Entrance Curve Design Speed (mph)				
	20	30	40	50	All Speeds
	3% ≤ G ≤ 4% Upgrade				3% ≤ G ≤ 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
	4% < G ≤ 6% Upgrade				4% < G ≤ 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

Notes:
1. *No adjustment is needed on grades less than 3 percent.*
2. *The grade in the table is the average grade measured over the distance for which the acceleration length applies. See Figures 10.4-H and 10.4-I.*

GRADE ADJUSTMENTS FOR ACCELERATION
(Passenger Cars)
Figure 10.4-K

* * * * * * * * * *

Example 10.4-2

Given:
Highway Design Speed = 70 miles per hour
Entrance Ramp Curve Design Speed = 40 miles per hour
Average Grade = 5 percent upgrade

Problem: Determine length of acceleration required.

Solution: Figure 10.4-J yields an acceleration length of 1000 feet on the level. According to Figure 10.4-K, this should be increased by a factor of 2.6.

Therefore: L = (1000)(2.6)
L = 2600 feet

Provide a 2600-foot acceleration length from the PT of the entrance ramp curve to the beginning of the taper.

* * * * * * * * *

Highway Design Speed (mph) (V)	Speed Reached (mph) (V_a)	L_a = Acceleration Length (ft)						
		For Entrance Curve Design Speed (mph)						
		Stop	15	20	25	30	35	40
		For Average Running Speed (mph) (V'_a)						
		0	14	18	22	26	30	36
55*	38	700	600	575	550	500	425	200
60	42	1300	1200	1175	1150	1100	1025	800
65	45	2100	2000	1975	1950	1900	1825	1600
70	48	2800	2700	2675	2650	2600	2525	2300

**For 55 miles per hour, the minimum lengths for passenger cars in Figure 10.4-J will apply.*

Notes:

1. *The acceleration lengths are calculated from the distance needed for a 200-pound per horsepower truck to accelerate from the average running speed of the entrance curve to reach a speed (V_a) that is 10 miles per hour below the average running speed on the mainline.*
2. *The taper entrance ramp is generally not applicable where trucks govern the design.*

LENGTHS FOR ACCELERATION
(200-Pound per Horsepower Truck)
Figure 10.4-L

10.4.2.4 Sight Distance

Provide drivers on the mainline approaching an entrance terminal sufficient distance to see the merging traffic so that they can adjust their speed or change lanes to allow the merging traffic to enter the freeway. Likewise, drivers on the entrance ramp need to see a sufficient distance upstream from the entrance to locate gaps in the traffic stream for merging. Therefore, provide decision sight distance according to the criteria in Section 4.3.

10.4.2.5 Superelevation

The entrance ramp superelevation should be gradually transitioned to meet the normal cross slope of the mainline. Apply the principles and criteria for superelevation as discussed in Section 5.3 to the entrance design. Section 10.4.1.5 provides the superelevation criteria for exit freeway/ramp junctions, which are also applicable to entrance freeway/ramp junctions. This includes e_{max}, design superelevation rate (e_d), transition lengths and point of revolution.

10.4.2.6 Cross Slope Rollover

The cross slope rollover is the algebraic difference between the slope of the through lane and the slope of the entrance ramp, where these two are adjacent to each other. The maximum algebraic difference is 5 percent beyond the physical nose. Between the gore nose and physical nose, the maximum cross slope rollover is 7 percent. See Section 10.4.2.8 for gore area definitions.

10.4.2.7 Shoulder Transitions

At entrance terminals, the right shoulder must be transitioned from the narrower ramp shoulder to the wider freeway shoulder (i.e., 6 feet to 10 feet). Figures 10.4-H and 10.4-I illustrate the typical shoulder transition. Note: Include taper rates for total shoulder width (paved and earth).

10.4.2.8 Gore Area

The following presents the nose dimensions for entrance gores:

1. <u>Painted Nose</u>. The painted nose dimension is considered to be 0.0 feet (i.e., the point where the two paint lines meet).
2. <u>Physical Nose</u>. The physical nose has a dimensional width of 10 feet, which is where the inside ramp edge meets the outside edge of the 10-foot paved freeway shoulder.
3. <u>Gore Nose</u>. The gore nose is where the outside edges of the ramp and mainline paved shoulders are 10 feet apart.

10.4.3 <u>Multilane Terminals</u>

Multilane terminals may be required when the capacity of the ramp is too great for single-lane operation. They may also be used to improve traffic operations (e.g., weaving) at the junction. The following lists several elements the designer should consider when a multilane terminal is required:

1. <u>Lane Balance</u>. Maintain lane balance at the freeway/ramp junction; see Section 10.3.4.
2. <u>Entrances</u>. For multilane entrance ramps, desirably use the parallel-lane design; however, a taper design may be considered.
3. <u>Exits</u>. For a multilane exit ramp, the additional lane should be at least 1500 feet prior to the terminal. The total length from the beginning of the first taper to the gore nose will range from 2500 feet for turning volumes of 1500 vehicles per hour or less up to 3500 feet for turning volumes of 3000 vehicles per hour.
4. <u>Signing</u>. Because of the complicated signing that may be required in advance of the exit, coordinate the geometric layout of multilane exits with the traffic designer.

10.4.4 Major Fork/Branch Connections

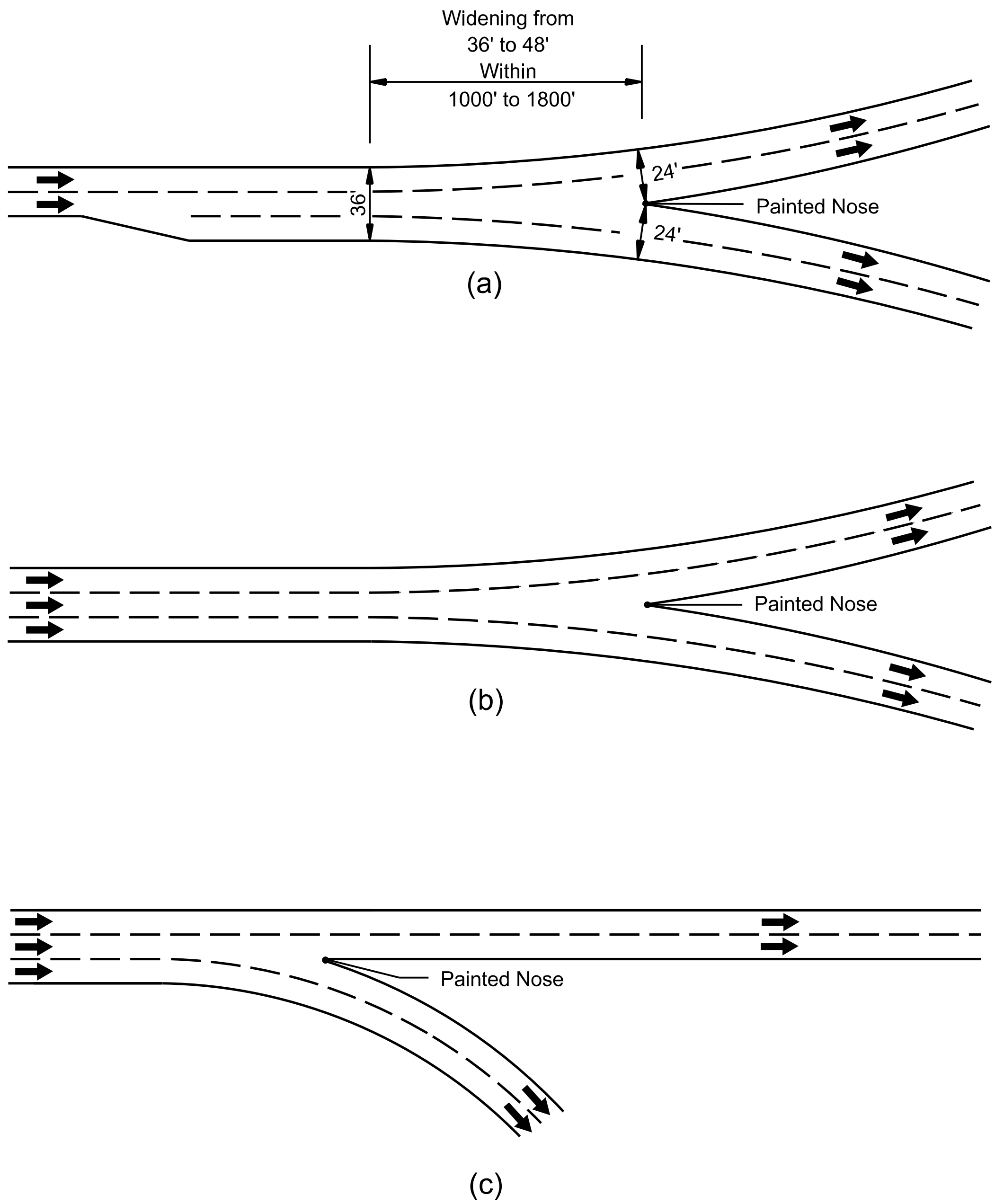
Figures 10.4-M and 10.4-N illustrate typical design details for a major fork or branch connection. The following presents a few geometric issues that the designer should consider when designing major divisions:

1. Lane Balance. Maintain the principle of lane balance; see Section 10.3.4.

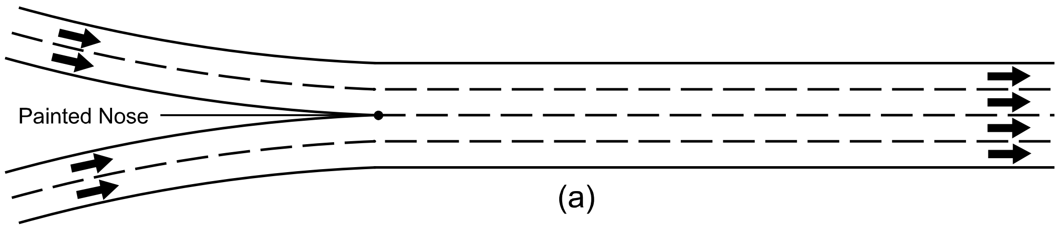
2. Divergence Point. Where the alignments of both roadways are on horizontal curves at a major fork, place the painted nose of the gore in direct alignment with the centerline of one of the interior lanes. This provides a driver in the center lane the option of going in either direction. See Figure 10.4-M(a) and (b). Where one of the roadways is on a tangent at a major fork, the gore design should be the same as a freeway/ramp multilane exit. See Figure 10.4-M(c).

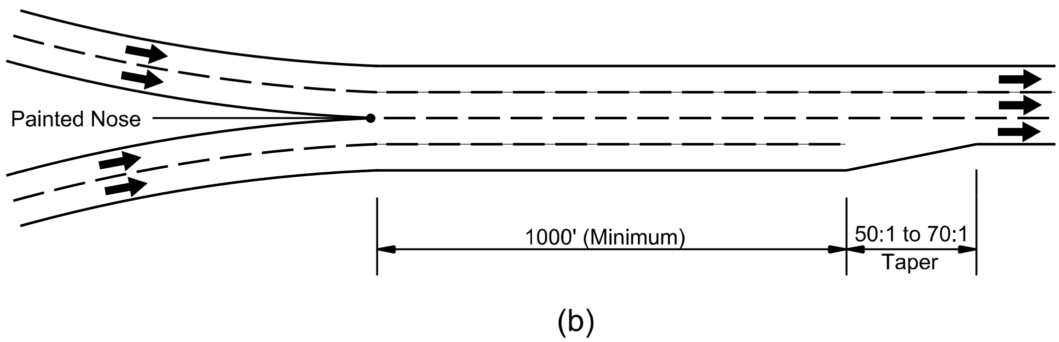
3. Nose Width. At the painted nose of a major fork, the lane should be at least 24 feet wide, but preferably not more than 28 feet. The widening from 12 feet to 24 feet should occur within a distance of 1000 feet to 1800 feet. See Figure 10.4-M(a).

4. Branch Connection. When merging, provide a full lane width for at least 1000 feet beyond the painted nose. See Figure 10.4-N(b).



MAJOR FORKS
Figure 10.4-M





BRANCH CONNECTIONS
Figure 10.4-N

10.5 RAMP DESIGN

For design purposes, the ramp proper is assumed to begin at the gore nose for exit ramps and end at the gore nose for entrance ramps.

10.5.1 Ramp Types

The components of a ramp include the freeway/ramp junction, the ramp proper, and a free-flow or controlled ramp terminal at the crossroad. Although ramps have varying shapes, each can be classified into one or more of the types as illustrated in Figure 10.5-A and discussed in the following sections.

10.5.1.1 Loop Ramps

There are two types of loop ramps:

1. Free-Flow. The free-flow loop, Figure 10.5-A(a), consists of compounded circular arcs that turn through approximately 270 degrees. The free-flow loop is a standard component of the cloverleaf interchange, four-quadrant partial cloverleaf interchange and trumpet interchange. Free-flow loops are designed so that either the central arc is a sharper radius than that of the initial and final arcs or the central arc is intermediate between the two. Motorists decelerate from the speed of the through highway over the initial portion of the ramp and accelerate uniformly over the final portion of the ramp.

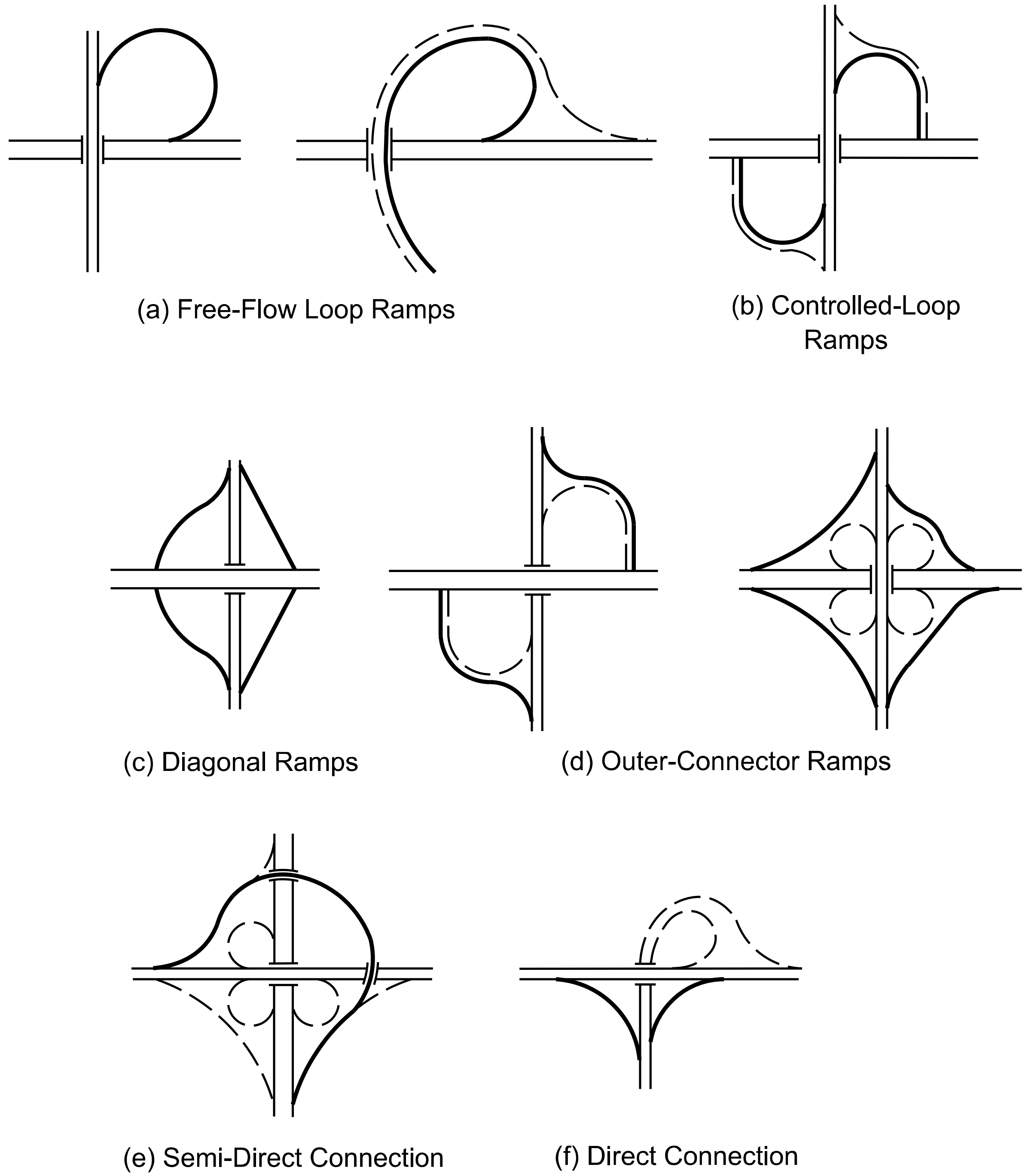
2. Controlled Terminal. Controlled terminal loops, Figure 10.5-A(b), are a component of the partial cloverleaf interchange. Controlled terminals are provided at the intersections with the crossroad and permit both right- and left-turning movements. Wherever practical, the angle of intersection should be 90 degrees.

10.5.1.2 Diagonal Ramps

Diagonal ramps, Figure 10.5-A(c), are components of the diamond interchange. Controlled terminals are provided on the crossroad. The angle of intersection with the crossroad varies between 60 degrees and 90 degrees.

10.5.1.3 Outer-Connector Ramps

Outer-connector ramps are in the same quadrant and to the outside of loop ramps; see Figure 10.5-A(d). They may have free-flow operation (e.g., at cloverleaf or trumpet interchanges) or have controlled operations (e.g., at partial cloverleaf interchanges).



Note: The heavier solid line indicates the ramp type being addressed.

RAMP TYPES
Figure 10.5-A

10.5.1.4 Semi-Direct Connections

Semi-direct connections are indirect in alignment, yet more direct than a loop ramp. These ramps are illustrated in Figure 10.5-A(e). Motorists making a left turn normally exit to the right and initially turn to the right, reversing direction before entering the intersecting highway. The outer ramp of the trumpet interchange is also a semi-direct connection.

10.5.1.5 Direct Connections

Direct connections do not deviate greatly from the intended direction of travel. These are illustrated in Figure 10.5-A(f) as an element of a trumpet interchange. They are also used to accommodate single-lane and right-turning traffic on four-quadrant partial cloverleafs and directional interchanges.

10.5.2 Design Speed

Figure 10.5-B provides the recommended ranges of ramp design speeds based on the design speed of the mainline. In addition, consider the following when selecting the ramp design speed:

1. Loop Ramps. Design speeds in the middle and upper ranges are generally not attainable for loop ramps. The following apply to loop ramps:

   a. For loop ramps on collector-distributor roadways or in restricted urban conditions, the minimum design speed for loops should be 25 miles per hour.

   b. Where the truck ADT is greater than 15 percent, use a minimum design speed of 30 miles per hour for the initial curve after the exit curve.

   c. For rural loop ramps, use a minimum design speed of 30 miles per hour.

   d. Use a design speed of 40 miles per hour for cloverleaf interchange loop ramps between freeways used in conjunction with C-D roads.

	Mainline Design Speed				
	55 mph	**60 mph**	**65 mph**	**70 mph**	**75 mph**
	Ramp Design Speed (mph)				
Upper Range	45-50	50	55	60	65
Middle Range	40	45	45	50	55
Lower Range	25-30	30	30	35	40

RAMP DESIGN SPEEDS
Figure 10.5-B

2. Outer Connector Ramps. The design speed for the outer connector ramp of a rural cloverleaf interchange should be 50 miles per hour. Where a wrap-around type ramp is used, use a minimum design speed of 45 miles per hour for the center curve.

3. Semi-Direct Connections. Use design speeds in the middle to upper ranges for semi-directional ramps. Do not use a design speed less than 30 miles per hour.

4. Direct Connections. These include both diagonal ramps at a diamond interchange and ramps at a directional interchange. Use a design speed in the middle to upper ranges. Do not use a design speed less than 40 miles per hour.

5. Controlled Terminals. If a ramp is terminated at an intersection with a stop or signal control, the design speeds in Figure 10.5-B are not applicable to the portion of the ramp near the intersection. The design speed on the ramp near the crossroad intersection can be a minimum of 25 miles per hour.

6. Variable Speeds. The ramp design speed may vary based on the two design speeds of the intersecting roadways (i.e., mainline design speed and crossroad design speed). The selected design speed should be consistent with the connecting facilities. When using multiple ramp design speeds, the maximum speed differential between controlling design elements (e.g., horizontal curves, vertical curves) should not be greater than 10 to 15 miles per hour. The designer must ensure that there is sufficient deceleration distance available between design elements with different design speeds (e.g., two horizontal curves). See Section 10.5.5.

Figure 10.5-C presents geometric design criteria for interchange ramps based on the selected design speed (e.g., sight distance, horizontal and vertical alignment).

10.5.3 Sight Distance

The designer should ensure that stopping sight distance is continuously provided along the ramp. Because ramps are composed of curves of various radii and design speeds, sight distance requirements may vary over the length of the ramp. The designer should provide decision sight distance at locations with complex maneuvers. Figure 10.5-C provides a summary of the geometric criteria for ramps, including stopping sight distance.

10.5.4 Cross Section Elements

Figure 10.5-D presents the typical cross sections for tangent and loop ramps. The following also applies to the ramp cross section:

1. Width. The total paved ramp width will be the sum of the ramp traveled way, the left shoulder and the right shoulder. For most ramps, the typical ramp traveled way is 16 feet. For locations with significant numbers of trucks and tight radii, consider widening the ramp shoulders. If the facility has unpaved shoulders, review the ramp shoulder criteria from the AASHTO *A Policy on Geometric Design of Highways and Streets* to determine the applicable ramp width. Assume the Case II and “C” design traffic conditions.

Geometric Requirements									
RAMP DESIGN SPEED (mph)	65	60	55	50	45	40	35	30	25
STOPPING SIGHT DISTANCE (ft)	645	570	495	425	360	305	250	200	155
HORIZONTAL ALIGNMENT									
Minimum Radius (ft) e_{max} = 8%	1480	1200	960	758	587	444	314	214	134
Minimum Length of Arc (ft)	See Figure 10.5-E								
VERTICAL ALIGNMENT									
Maximum Grades	3-5%	3-5%	3-5%	3-5%	3-5%	4-6%	4-6%	5-7%	5-7%
Crest Vertical Curves (K-values)*	193	151	114	84	61	44	29	19	12
Sag Vertical Curves (K-values)*	157	136	115	96	79	64	49	37	26

**K-values are based on stopping sight distance on level grades.*

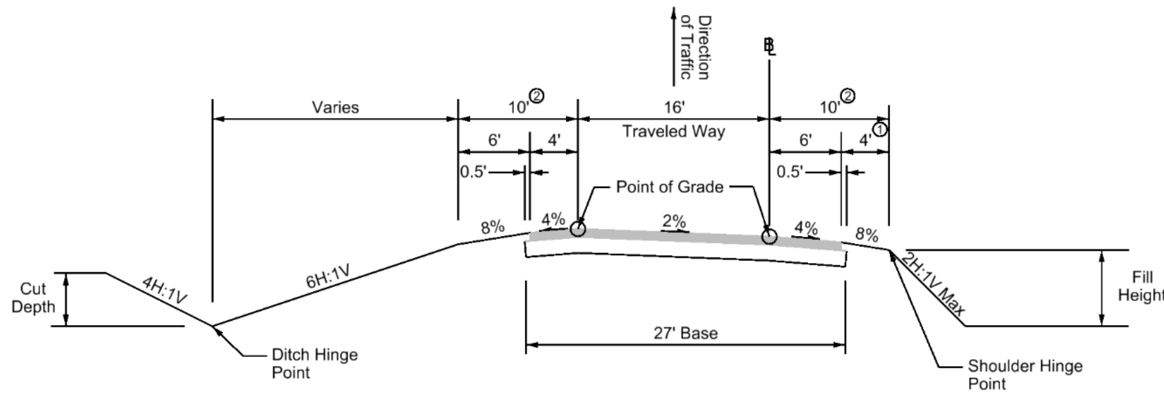
ALIGNMENT CRITERIA FOR INTERCHANGE RAMPS
Figure 10.5-C

The typical right-paved shoulder is 6 feet, and the typical left-paved shoulder is 4 feet in the direction of travel. The left shoulder-traveled way-right shoulder arrangement is illustrated in the ramp cross sections in Figure 10.5-D. For multilane directional ramps, the cross sectional width is the same as the mainline design (e.g., 24-foot traveled way width plus shoulders); see Chapter 17 “Freeways.”

2. Cross Slope. For tangent sections, the ramp traveled way is sloped unidirectional at 2.00 percent towards the right shoulder. Shoulder cross slopes on tangent are typically 4.00 percent. The left shoulder is typically sloped away from the traveled way.

3. Curbs. Only use curbs on urban interchange ramps and only where necessary. If curb and gutter is required for drainage, use sloping curb and place it on the outside edge of the full-width paved shoulders. See Section 7.2.8 for information on the use of curbs.

4. Bridges and Underpasses. Carry the full width of the ramp (shoulders and travel lanes), across the bridge. See Chapters 7 “Cross Section Elements” and 17 “Freeways” when determining the clear ramp width for an underpass.



Notes:

① *Add 3.75 feet where guardrail is used.*

② See Section 5.3 for maximum shoulder break.

TYPICAL RAMP CROSS SECTIONS
Figure 10.5-D

5. Side Slopes/Ditches. For the ramp proper, side slopes and ditches should meet the same criteria as for the highway mainline. Chapters 7 "Cross Section Elements" and 17 "Freeways" provide the applicable design information for side slopes and ditches.

6. Roadside Safety. See the AASHTO *Roadside Design Guide* for clear zone criteria and barrier warrants, selection and layout.

7. Right of Way. The right of way adjacent to the ramp is fully access controlled and the right of way is typically fenced. See Chapter 12 "Right of Way."

10.5.5 Horizontal Alignment

The following will apply to the horizontal alignment of ramps:

1. Minimum Curve Radii. Figure 10.5-C provides the minimum curve radii based on ramp design speed and e_{max}.

2. Superelevation Rates. See Section 5.3 for superelevation rates based on design speed, design superelevation and curve radius.

3. Curve Type. On all ramps, except loop ramps, only use simple curves unless field constraints (e.g., to avoid an obstruction) dictate the use of compound curvature. On loop ramps, the designer should typically use compound curves with the interior curve(s) of sharper radii than the exterior curves. For exits with loops, the radii of the flatter arc

compared to the radii of the sharper arc should not exceed a ratio of 2:1 to prevent abruptness in operation and appearance. Where compound arcs of decreasing radii are used, the arcs should have sufficient length to enable motorists to decelerate at a reasonable rate over the range of design speeds. See Figure 10.5-E.

Comparable radii and length controls may be used on entrance loop ramps with compound arcs of increasing radii. However, for entrance ramps, the 2:1 ratio of compound curves and the lengths in Figure 10.5-E is not as critical because the vehicle is accelerating into a curve with a larger radius or into a tangent section.

Radius (ft)	**100**	**150**	**200**	**250**	**300**	**400**	**500 or more**
Minimum (ft)	40	50	60	80	100	120	140
Desirable (ft)	60	70	90	120	140	180	200

Note: These lengths are applicable to ramp curves followed by a curve 1/2 its radius or preceded by a curve of double its radius.

ARC LENGTHS FOR COMPOUND CURVES
Figure 10.5-E

4. Trucks. Where there are a significant number of trucks on loop ramps, the designer should consider how the design may impact the rollover potential for large trucks. To reduce this potential, consider using flatter curve radii and/or a higher ramp design speed than the allowable minimums. Other critical factors include ensuring that ample deceleration lengths are available and, if judged necessary by the traffic designer, include special rollover warning signs for trucks.

5. Baseline/Centerline. The following will apply:

   a. Ramp. Typically, the outside edge (away from the interior of the interchange) of the ramp traveled way is used for horizontal and vertical control. This edge may be used for the point of grade/revolution or the designer may choose to use the inside edge.

   b. Loop. Typically, the inside edge (near the interior of the interchange) of the loop traveled way is used for horizontal and vertical control. This edge may be used for the point of grade/revolution or the designer may choose to use the outside edge.

6. Controlled Ramp Termini. Exit ramps may end at a controlled intersection — stop control or signal control. See Chapter 9 “Intersections.”

10.5.6 Vertical Alignment

10.5.6.1 Grades

Maximum grades for vertical alignment cannot be as definitively expressed as those for the highway mainline. General values of limiting gradient are shown Figure 10.5-C, but for any one ramp the selected profile is dependent upon a number of factors. These factors include:

1. The flatter the gradient on the ramp, the longer the ramp will be. At restricted sites (e.g., loops), it may be advantageous to provide a steeper grade to shorten the ramp length.

2. Use the steepest gradients for the center portion of the ramp. Freeway/ramp junctions and landing areas at intersections should be as flat as practical.

3. Short upgrades up to 5 percent do not unduly interfere with truck and bus operations. Consequently, for new construction it is desirable to limit the maximum gradient to 5 percent.

4. Downgrades on ramps should follow the same guidelines as upgrades. However, where there are sharp horizontal curves and significant truck and bus traffic, it is desirable to limit the downgrades to 3 to 4 percent.

5. The ramp grade within the freeway/ramp junction up to the physical nose should be approximately the same grade as that provided on the mainline.

10.5.6.2 Vertical Curvature

Design vertical curves on ramps to meet the stopping sight distance criteria based on the ramp design speed as presented in Section 6.5. Figure 10.5-C provides the K-values for both crest and sag vertical curves on level grades. The ramp profile often assumes the shape of the letter S with a sag vertical curve at one end and a crest vertical curve at the other. In addition, where a vertical curve extends onto the freeway/ramp junction, determine the length of curve using a design speed comparable to the mainline. See Section 6.5 for details on the design of vertical curves.

10.5.6.3 Cross Sections Between Adjacent Ramps

Where the alignment of a ramp is designed to be parallel to an adjacent ramp (e.g., cloverleaf, trumpet interchanges), first establish the profile of the loop ramp and then set the profile of the outer ramp to be approximately parallel to the inner-loop ramp profile. Accomplish this by calculating the left-edge elevations of the loop ramp and matching those elevations for the left-edge elevations of the outer ramp.

10.5.7 Ramp Merges

The designer needs to give special consideration to the design where two ramps merge to together (e.g., directional interchange ramps), which are designed on a case-by-case basis. These merges will require coordination with the traffic designer to conduct an operational analysis.

10.6 RAMP/CROSSROAD INTERSECTION

Chapter 9 "Intersections" presents the Department's in-depth criteria on the design of at-grade intersections. This section presents additional information that is applicable to the intersection of an interchange ramp and the crossroad.

At diamond and partial cloverleaf interchanges, the ramp will terminate or begin with an at-grade intersection. In general, design the intersection as described in Chapter 9. This will involve a consideration of capacity and the physical geometric design elements (e.g., sight distance, angle of intersection, acceleration lanes, channelization and turning lanes). The designer should also consider the following in the design of the ramp/crossroad intersection:

1. Crossroad Width. The crossroad width will be based on the anticipated traffic volumes for the design year, the crossroad functional classification and the design criteria presented in Chapter 9 "Intersections."

2. Sight Distance. Section 4.4 discusses the criteria for intersection sight distance (ISD). Ramp/crossroad intersections present unique ISD problems because of the nearby bridge structure at most interchanges. Give special consideration to the location of bridge piers, abutments, sidewalks, bridge rails, roadside barriers, etc.; these elements may present major sight obstructions. The bridge obstruction and the required ISD may result in the relocation of the ramp/crossroad intersection further from the structure. Also, crest vertical curves on the crossroad may need to be flattened to provide adequate sight distance in the vertical plane.

3. Capacity. In urban areas where traffic volumes are often high, inadequate capacity of the ramp/crossroad intersection can adversely affect the operation of the ramp/freeway junction. In a worst-case situation, a backup onto the freeway may impair the safety and operation of the mainline itself. Therefore, give special attention to providing sufficient capacity and storage for an at-grade intersection or a merge with the crossing road. This may require providing additional lanes at the intersection or on the ramp proper, or it could involve a specialized design to reduce queuing. The analysis must also consider the operational impacts of the traffic characteristics in either direction on the intersecting road.

4. Turn Lanes. Exclusive left- and/or right-turn lanes often will be required on the crossroad and in many cases on the ramp itself. Chapter 9 "Intersections" provides information on the design of turn lanes at intersections.

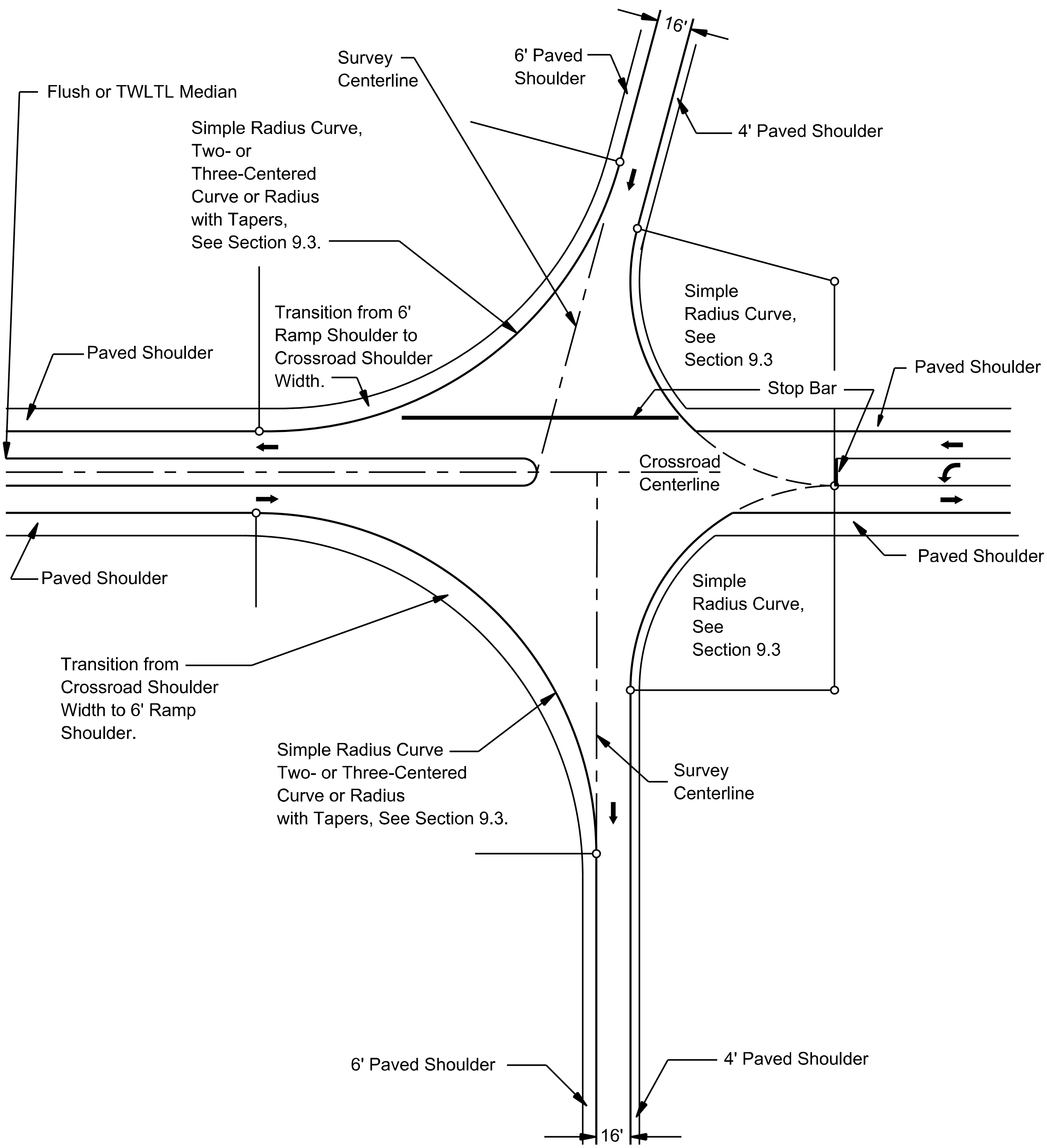
5. Signalization. Where queuing at one intersection is long enough to affect operations at another, the two intersections may require a larger separation or coordinated signal design.

6. Design Vehicle. Design all radius returns and left-turn control radii for ramp/crossroad intersections using a WB-62 design vehicle; see Section 9.3. Use the WB-67 design vehicle for determining storage lengths (e.g., left-turn lanes), median widths, etc., for the ramp/crossroad intersection.

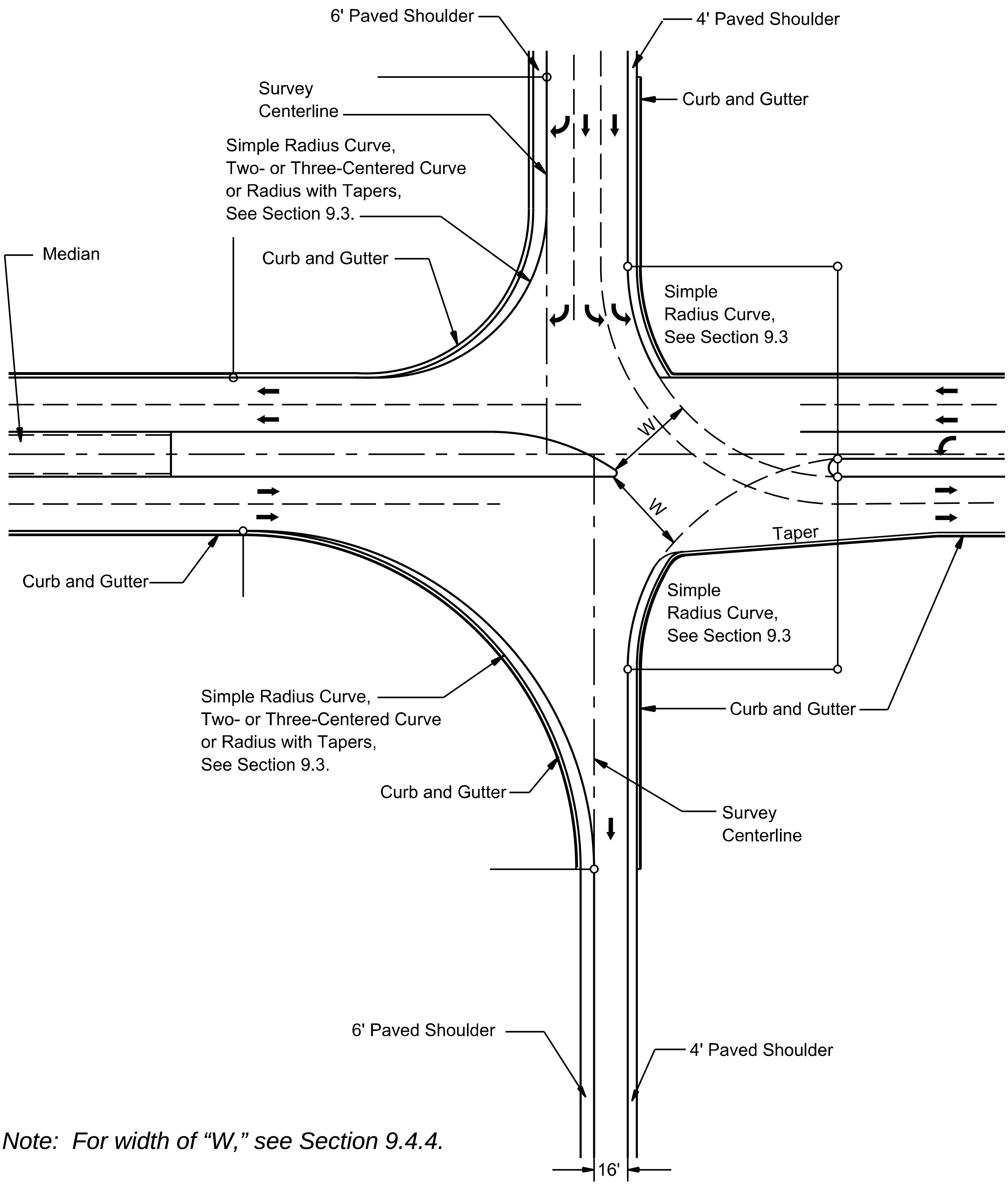
7. Typical Designs. Figures 10.6-A and 10.6-B illustrate typical ramp/crossroad intersections for a diamond interchange. Figure 10.6-A illustrates a three-lane crossroad and Figure 10.6-B a four-lane multilane curb and gutter crossroad with a two-way, left-turn lane.

8. Wrong-Way Movements. Wrong-way movements often originate at the ramp/ crossroad intersection onto an exit ramp. To minimize their probability, design the intersection geometry to discourage this movement and provide appropriate access management techniques (e.g., raised medians).

9. Access Control. Section 3.8 presents detailed information on access control along the crossroad in the vicinity of ramp/crossroad intersections.



RAMP/CROSSROAD INTERSECTION—DIAMOND INTERCHANGE
(Three-Lane Crossroad)
Figure 10.6-A



Note: For width of "W," see Section 9.4.4.

RAMP/CROSSROAD INTERSECTION — DIAMOND INTERCHANGE
(Five-Lane TWLTL Crossroad — Signalized Intersections)
Figure 10.6-B

10.7 REFERENCES

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