Chapter Twenty-nine
INTERCHANGES

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Chapter Twenty-nine
INTERCHANGES

29.1 GENERAL

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. Chapter Twenty-nine provides guidance in the design of interchanges including access guidelines, selection, operations, spacing, freeway/ramp terminals, ramps and ramp/crossroad terminals.

29.1.1 Responsibility

The following units are responsible for the planning and design of an interchange:

1. Access Review. The Rail, Transit and Planning Division is responsible for requests to FHWA for new Interstate access.

2. Interchange Type Selection. Once it has been determined that an interchange is justified, a traffic engineering study will determine the appropriate interchange type for the site.

3. Geometric Layout. The traffic engineering study determines the interchange layout including the horizontal alignment, the preliminary profile grade line, ramp/crossroad intersection details and appropriate traffic control.

4. Interchange Design. On the basis of the traffic engineering study, the road design process determines the final vertical alignment, earthwork quantities, drainage design and contour grading plans. In addition, the Road Design Section will coordinate with the Right-of-Way Bureau and Traffic Engineering Section to determine the necessary access control and right-of-way limits.

5. Detailed Sheets. The Road Design Section in coordination with the Geometrics Unit will be responsible for preparing the detailed sheets that will be included in the construction plans.

6. Consultant Projects. On consultant-designed interchange projects, the consultant will be responsible for the design of all elements including type selection, geometric layout, signing, electrical work, ramp/crossroad intersection

details and detailed plan preparation. The Traffic Engineering Section will be responsible for reviewing these items.

29.1.2 Guidelines

The high cost and environmental impact require that interchanges be used only after careful consideration of its merits. Because of the great variance in specific site conditions, MDT has not adopted specific interchange warrants. When determining the need for an interchange or grade separation, consider the following:

1. Fully Access-Controlled Facilities. On fully access-controlled facilities, each intersecting highway must be terminated, rerouted, provided a grade separation or provided an interchange. The importance of the continuity of the crossing road, the feasibility of an alternative route, traffic volumes, construction costs, environmental impacts, etc., will determine the purpose and need for a grade separation or interchange. An interchange should be provided on the basis of the purpose for access and anticipated demand for access by the freeway user. An additional access factor is in areas where access availability from other sources is not practical. In these cases, access to the freeway may be the only reasonable option.

2. Limited Access-Controlled Facilities. On facilities with limited control of access, intersections with public roads will normally be accommodated by at-grade intersections. In general, it is rare that a grade separation or an interchange will be provided as an alternative because the free-flow intent of an interchange is difficult to provide when the facilities are not access controlled. Topography, in most cases, limits the potential to provide grade separation, and costs are generally prohibitive. However, an interchange may still be a viable option for high-volume intersecting roads when considering the following:

    a. the intersection is between two major facilities (e.g., arterial to arterial);

    b. the level-of-service (LOS) at the intersection is unacceptable, and the intersection cannot be redesigned at-grade to operate at an acceptable LOS;

    c. the site topography is adaptable to an interchange;

    d. the interchange addresses the safety concerns of the at-grade intersection; and/or

    e. the facility can be upgraded to support a freeway alignment at the desired point of access.

29.1.3 New/Reconstructed Interchanges

MDT's goal is to maintain the highest practical level of service, safety and mobility on its Interstate System. Among other design features, this is accomplished by controlling access onto the system. In general, MDT and FHWA discourage new access points on existing, fully access-controlled facilities.

29.1.3.1 Changes in Access

Each entrance and exit point on the mainline, including "locked-gate" access (e.g., utility opening), is defined as an access point (e.g., diamond interchanges have four access points). A revised access is considered to be a change in the interchange configuration even though the number of access points may not change (e.g., replacing a diamond interchange ramp with a loop).

The designer 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.

29.1.3.2 Processing Procedures

MDT 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. 55, No. 204, Monday, 10-22-90 and Federal Register, Vol. 63, No. 28, Wednesday, 2-11-98. Section 29.1.3.3 summarizes the criteria from the Federal Register.

The following procedures are applicable where 1) the highway is on the State Highway System and Federal funds were used for right of way and/or construction costs of the roadway segment; and 2) the highway is access controlled and the proposed access revisions will modify previous commitments made in environmental documents:

1. Environmental Study Determination. The Environmental Bureau will determine the type and scope of the necessary environmental process in cooperation with FHWA. Depending upon the magnitude of the proposed changes, a revision in access control may create impacts that require discussion in the appropriate environmental document.

2. Secondary Impacts. Determine the secondary impacts associated with the proposed access revisions based on traffic-induced impacts on the highway

facility and the potential environmental impacts on the surrounding area. Because the area of influence on the highway facilities and surrounding land use will vary, describe the limits of influence for each case prior to determining impacts.

3. FHWA Coordination. The Rail, Transit and Planning Division, in conjunction with the Traffic Engineering Section, usually will review and approve the interchange type and interchange design. For Interstates and NHS projects, FHWA must also approve the interchange type and design details.

29.1.3.3 Documentation of Requests

All requests for changes in access must include the following:

1. Traffic Volumes. A capacity analysis must be performed to determine if an existing facility does not provide the necessary access nor can it be improved to accommodate the expected design year LOS.

2. Alternatives. The request must demonstrate that all reasonable alternatives for design options, locations and transportation system management (TSM) type improvements (e.g., ramp metering, mass transit, HOV facilities) have been evaluated, provided for and/or provision made for future incorporation.

3. Impacts. The proposed new access point must not have a significant adverse impact on the safety and operation of the freeway facility based on an analysis of current and future traffic (e.g., 20 years in the future). The investigations of traffic operations related to existing conditions should include:

   a. an analysis of adjacent freeway sections including the distance to the next interchange in each direction; and

   b. an analysis of crossroads and other roads/streets to ensure their ability to collect and distribute traffic to and from the proposed interchange.

   The request must demonstrate if acceptable merge and diverge lengths are available and if adequate signing can be provided.

4. Connections. The proposed new interchange should only connect to a public road and must provide for all traffic movements. Less than "full interchanges" will only be considered for special purposes and will be determined on a case-by-case basis.

5. Land Use. The request must evaluate the consistency between the interchange and local and regional development plans and transportation system improvements. Include information on the distance to and size of communities directly served by the interchange. For possible multiple interchange additions, a comprehensive Interstate/freeway network study that addresses all proposed and desired access within the context of a long-term plan must support the proposal.

6. New/Expanded Developments. Where new or revised access is requested due to proposed or expanded development, document that appropriate coordination has taken place with the developer in conjunction with other transportation system improvements. Prior to the new access being approved, confirm that a sufficient analysis of parallel facilities and crossroads are complete to ensure that local roads can adequately accommodate the additional traffic volumes.

7. Design. Ensure that the Department's design criteria for interchanges have been met or are adequately addressed.

8. Planning/Environmental. Ensure that the request for new or revised access contains information relative to the planning requirements and the status of the environmental processing of the proposal.

9. Capacity Analyses. Include the following information with the capacity analyses:

    a. current and future design hour traffic volumes for mainline traffic and for each ramp movement;

    b. current and proposed basic information including the number of lanes on the mainline and for each ramp, the distances in each direction between proposed ramps, and existing ramps on adjacent interchanges;

    c. layout (including number of lanes) for ramp intersections with crossroad and LOS;

    d. information on the terrain, either in general terms or, if necessary, specific grades if they affect the operations in the area;

    e. overall peak-hour factor and percentage of trucks on the mainline and on each ramp; and

    f. current and proposed signal phasing and pedestrian volumes at signalized intersections with the crossroad and ramps.

29.1.4 Grade Separation Versus Interchange

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 a grade separation or 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. Interchanges to other highways should be provided 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. Spacing of urban interchanges between interchange crossroads should not be less than 1 mi (1.5 km). This allows a minimum distance for an entering driver to adjust to the freeway environment, for weaving maneuvers between entrance and exit ramps, and for adequate advance and turnoff signing. Where the interchange spacing is less than 1 mi (1.5 km), consider providing collector-distributor roads or auxiliary lanes to compensate for the large differential in running speeds that can result in the through lanes. On the Interstate system in rural areas, do not locate interchanges less than 2 mi (3 km) apart.

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 capacity of the freeway exit and crossroad entrance.

29.2 INTERCHANGE TYPE SELECTION

29.2.1 Types

This section describes the basic types of interchanges. Section 29.2.2 discusses the selection of the interchange type. Each interchange must be custom designed to fit the individual site. 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.

29.2.1.1 Conventional Diamond Interchange

The conventional diamond is the simplest and the most common type of interchange. One-way diagonal ramps are provided in each quadrant with two at-grade intersections provided at the minor road. If these two intersections can be properly designed (e.g., intersection capacity, adequate storage distance between ramps, vertical and horizontal alignment), the diamond is usually the best choice of interchange where the intersecting road is not access controlled. Figure 29.2A illustrates a typical diamond interchange without frontage roads. Figure 29.2B illustrates a typical diamond interchange with frontage roads.

Some of the advantages and disadvantages of a conventional diamond interchange include:

Advantages

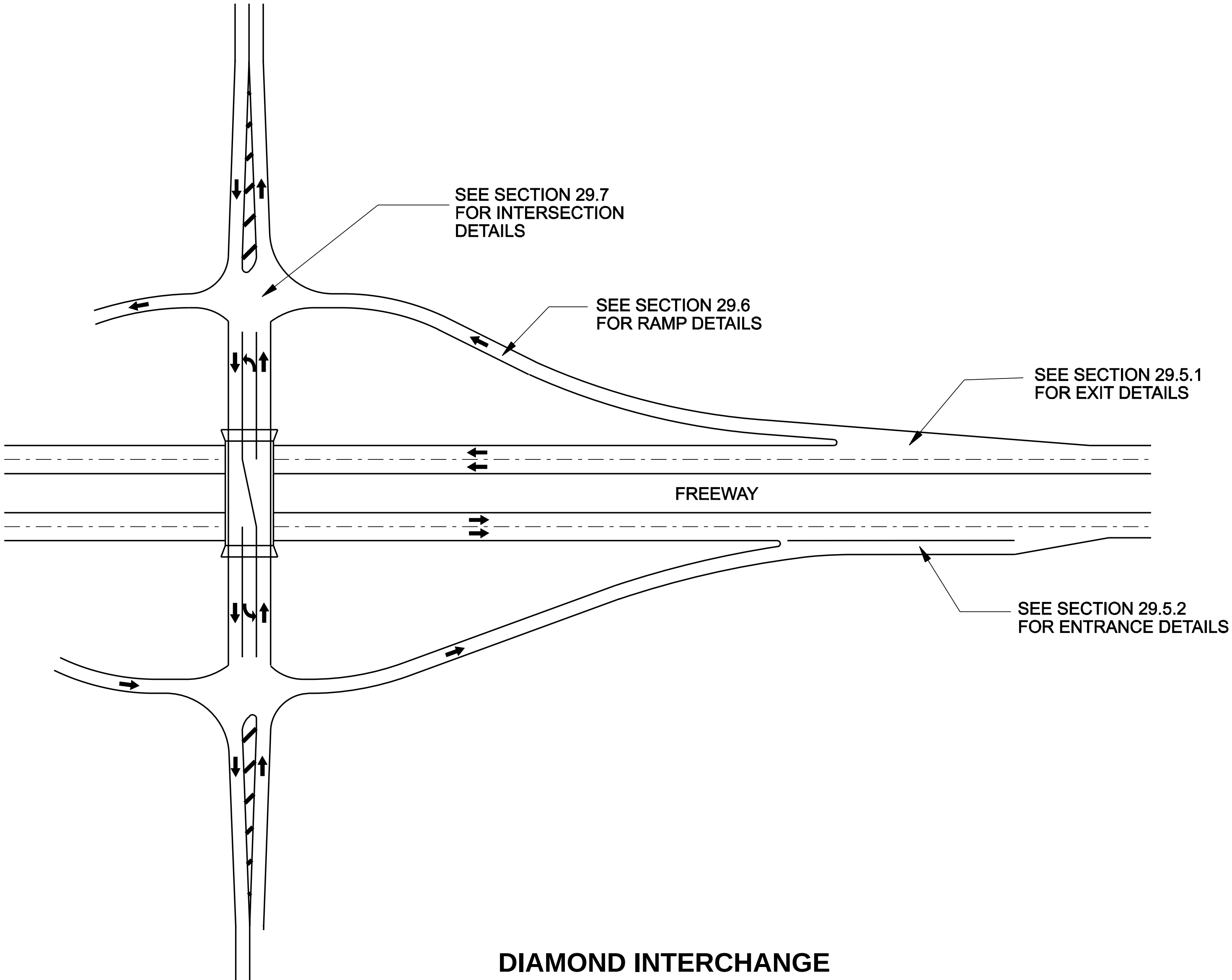
1. All exits from the mainline are made before reaching the crossroad structure and entrances occur after the structure. This allows good exit visibility and conforms to driver expectancy thereby minimizing confusion.

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

3. The operational maneuvers at the crossroad are consistent with other at-grade intersections on the crossroad.

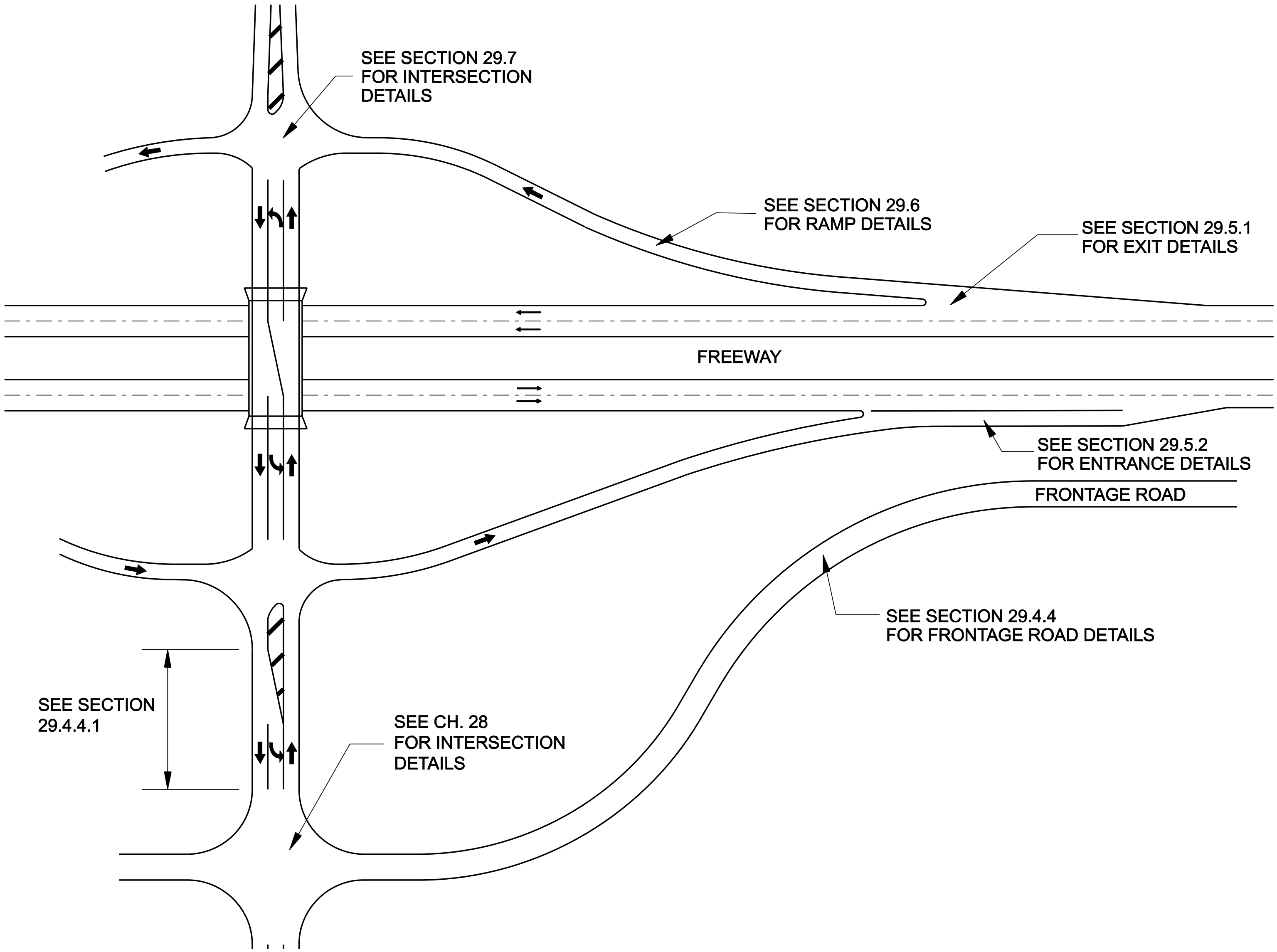
4. Left-turning maneuvers at the crossroad require little extra travel distance.

5. The diamond configuration easily allows modifications to provide greater ramp capacity, if needed in the future.



DIAMOND INTERCHANGE
(Without Frontage Roads)

Figure 29.2A



DIAMOND INTERCHANGE
(With Frontage Roads)

Figure 29.2B

6. Their common usage has resulted in a high degree of driver familiarity.

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

<u>Disadvantages</u>

1. There are potential operational problems with the two at-grade intersections at the crossroad (e.g., sight distance, left-turn storage between ramps, signal coordination).

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

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

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

5. The diamond requires right-of-way in all four quadrants of the interchange.

29.2.1.2 Compressed Diamond Interchange

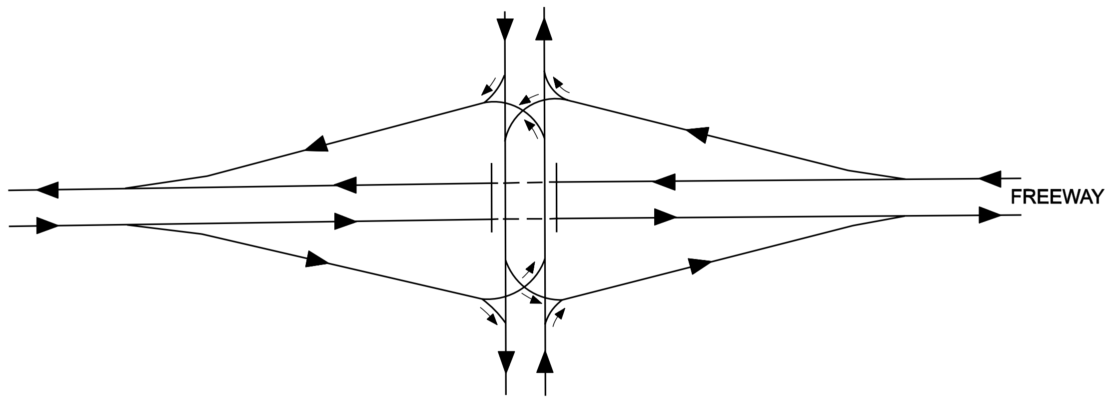
A compressed diamond interchange, also called a tight diamond interchange, is similar to the conventional diamond except that the ramp termini on the crossroad are located near the structure. Figure 29.2C illustrates a schematic of a compressed diamond interchange. This design type is generally only 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.

Some of the advantages and disadvantages of the compressed diamond include:

<u>Advantages</u>

1. 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.



COMPRESSED DIAMOND INTERCHANGE

Figure 29.2C

3. The crossing structure is significantly smaller than that for a single-point diamond, retaining walls are less expensive and construction costs are generally lower.

4. The ramp termini operate as two typical intersections, similar to a conventional diamond, and therefore are less confusing to drivers. If traffic signal control is justified, a single controller can be deployed to control both ramp termini.

<u>Disadvantages</u>

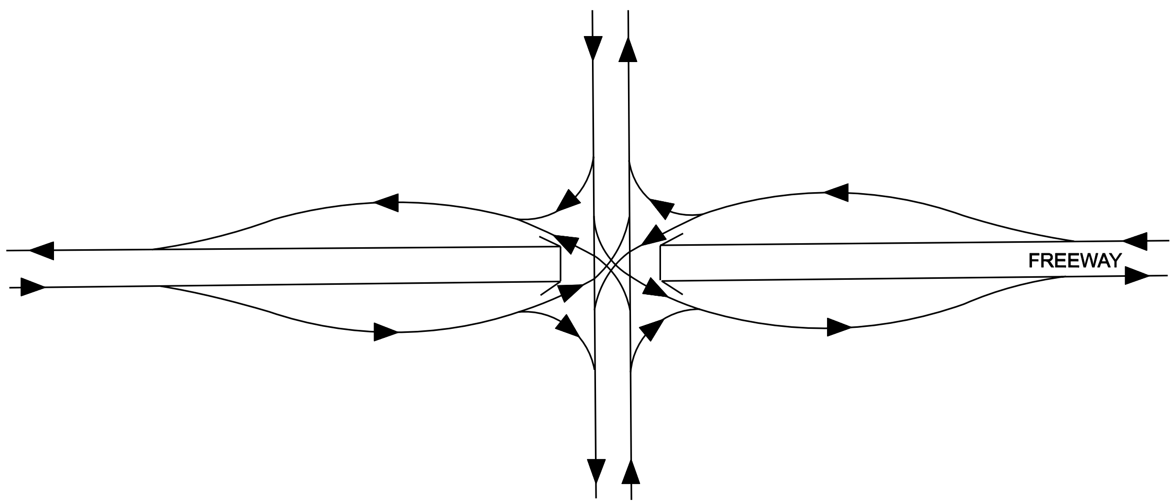
1. Left-turn lanes between the ramp termini may need to be overlapped (i.e., side-by-side opposing left-turn lanes). Consequently, the cross section of the crossroad is wider requiring longer or wider structures. Left-turn lanes may extend beyond the outside of the interchange to satisfy storage requirements.

2. Coordinated operation of the two ramp termini is required to prevent overlapping traffic queues from causing blockages.

3. Due to the close proximity of the two intersections, the compressed diamond typically will need to operate as a six-phase, overlap signal system under the control of a single controller. Typically, additional overlaps must be wired into the controller cabinet.

29.2.1.3 Single-Point Urban Interchange

The single-point urban interchange (SPUI) offers improved traffic-carrying capabilities, safer operations and reduced right-of-way needs when compared with other interchange configurations. It is typically considered in urban environments where right-of-way costs can exceed the cost of construction. The distinguishing feature of this interchange is the convergence of all through and left-turn movements into a single, signalized intersection. Figure 29.2D illustrates a schematic of a single-point urban interchange.



SINGLE-POINT URBAN INTERCHANGE

Figure 29.2D

Some of the advantages and disadvantages of this interchange type include:

<u>Advantages</u>

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

2. The SPUI can increase interchange capacity and alleviate storage problems that result from two closely spaced intersections on the crossroad. In particular, it overcomes the left-turn lane storage problem for drivers wishing to enter the freeway.

3. It only requires one signalized intersection instead of potentially two at a typical diamond.

4. Opposing left turns operate to the left of each other; therefore, their paths do not conflict.

Disadvantages

1. The SPUI presents a significantly larger intersection area for pedestrians to cross.

2. Because of the larger intersection area, it requires longer signal clearance intervals than a conventional diamond.

3. It has a higher cost than the typical diamond because of the need for a long, single-span structure and the need for retaining walls along the mainline.

4. Where the mainline crosses over the crossroad, lighting is required under the structure.

5. Special treatment of the traffic signal design is required (e.g., signal head placement).

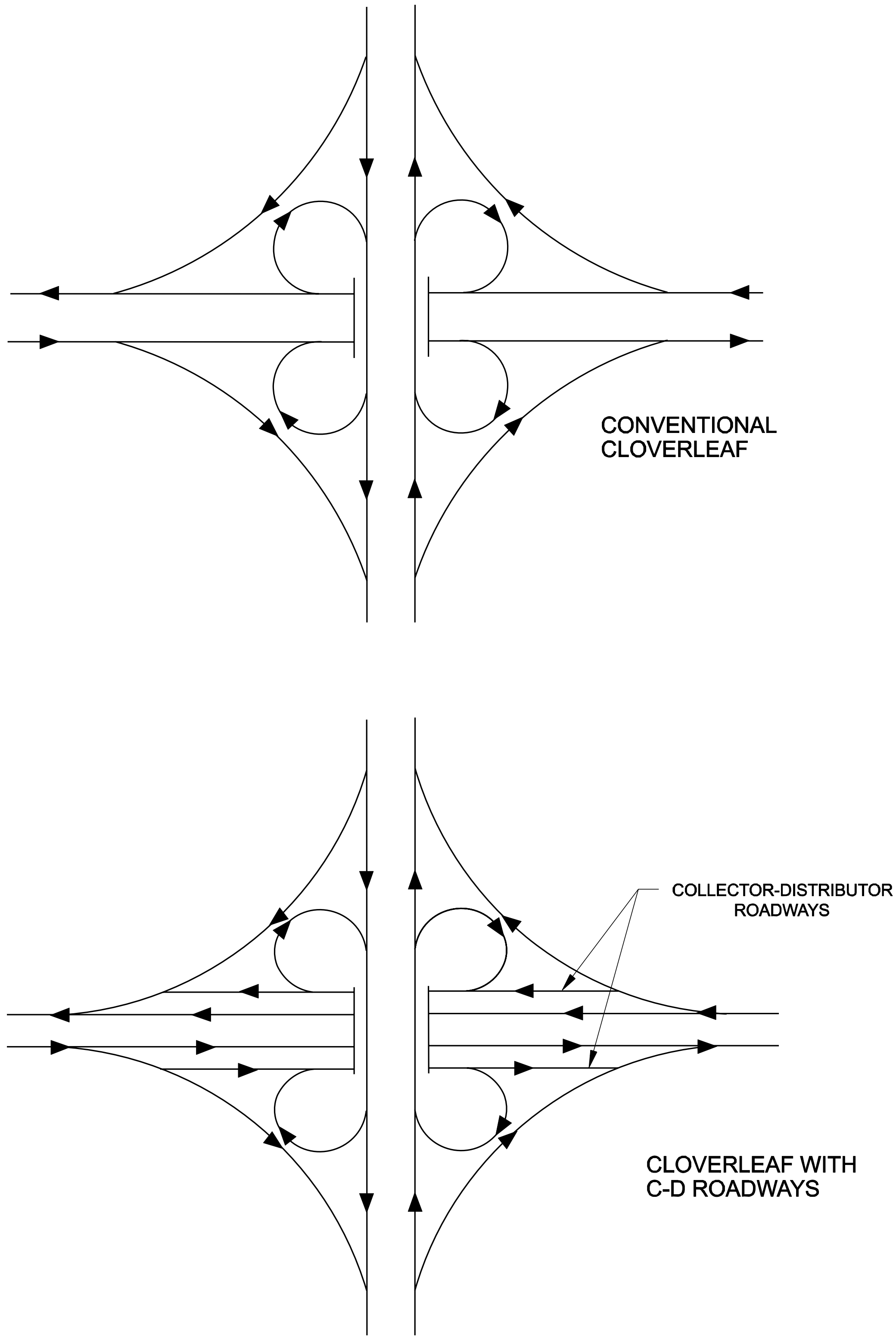
29.2.1.4 Full Cloverleaf Interchanges

Cloverleaf interchanges are used at 4-leg intersections and employ loop ramps to accommodate left-turn movements. Loops may be provided in any number of quadrants. Full cloverleaf interchanges are those with loops in all four quadrants; all others are partial cloverleafs.

Where two access controlled highways intersect, a full cloverleaf is the minimum type of interchange design that will suffice. They are not considered appropriate where both roadways are not fully access controlled. Figure 29.2E illustrates a cloverleaf with and without collector-distributor roads.

Cloverleafs introduce several undesirable operational features including, for example, double exits and entrances, weaving between entering and exiting vehicles and, when compared to directional interchanges, additional travel time and distance for left-turning vehicles. The operation of a cloverleaf is greatly improved through the addition of collector-distributor roadways; see Section 29.4.3.

Some of the advantages and disadvantages of full cloverleaf interchanges include:



FULL CLOVERLEAF INTERCHANGES

Figure 29.2E

Advantages

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

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

Disadvantages

1. Full cloverleafs require large amounts of right-of-way for the geometric design of loops.

2. They are typically more expensive than diamonds due to longer ramp lengths, wider structures and, if provided, the additional cost of collector-distributor roadways.

3. Without collector-distributor roadways, half the exits and entrances are located beyond the crossroad structure, which does not conform to driver expectancy. This requires additional signing to guide motorists.

4. Weaving sections in cloverleafs must be made long enough to provide for satisfactory traffic operations.

5. 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 characterizes the design of cloverleafs:

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

2. Loop Radii. 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 requirement on the downstream end is essential.

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

4. Loop Capacity. Expected design capacities for single-lane loops range from 800 vph to 1200 vph. The higher volumes are generally only achievable where the design speed is 30 mph (50 km/h) or higher and few trucks use the loop.

5. Weaving Volumes. 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 vph, interference increases rapidly with a resulting reduction of the through traffic speed and level of service. At these weaving volume levels, consider providing collector-distributor roadways.

6. Weaving Lengths. 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 ft (300 m) or the distance determined by a capacity analysis, whichever is greater.

7. Collector-Distributor Roadways. Providing 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. The lower speed weaving area is easier to match with the loop design.

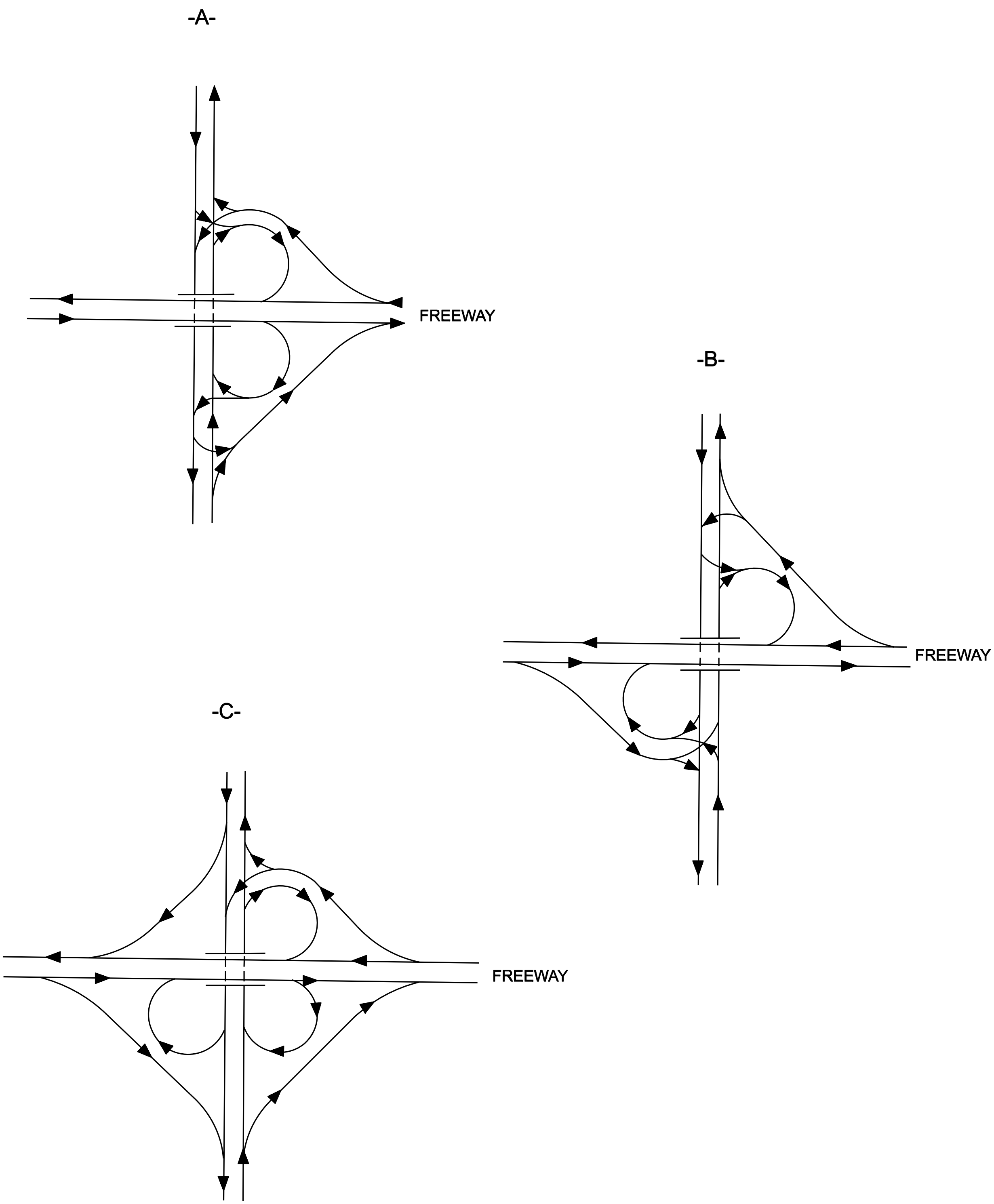
29.2.1.5 Partial Cloverleaf Interchanges

Partial cloverleaf interchanges are those with loops in one, two or three quadrants. They are appropriate where right-of-way restrictions preclude ramps in one or more quadrants. They are also advantageous where a left-turn movement can be provided onto the major road by a loop without the immediate presence of an entrance loop from the minor road. Figure 29.2F illustrates several examples of partial cloverleafs.

Several of the advantages and disadvantages listed for full cloverleafs also apply to partial cloverleafs (e.g., geometric restriction of loops). Some specific advantages of partial cloverleafs include:

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

2. Partial cloverleafs are often appropriate where one or more quadrants present adverse right-of-way and/or terrain problems that preclude using a diamond interchange.



PARTIAL CLOVERLEAF INTERCHANGES

Figure 29.2F

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

4. In some cases, they can reduce the width of structures.

29.2.1.6 Three-Leg Interchanges

Three-leg interchanges, also known as T- or Y-interchanges, are provided at intersections with three legs. Figure 29.2G illustrates examples of three-leg interchanges with several methods of providing the turning movements. The trumpet type is shown in (A) where three of the turning movements are accommodated with direct or semi-direct ramps and one movement by a loop ramp. In general, the semi-direct ramp should favor the heavier left-turn movement and the loop the lighter volume. Examples (B) and (C) are options to be considered when space is limited and/or design traffic volumes are moderate.

29.2.1.7 Directional and Semi-Directional Interchanges

The following definitions apply to directional and semi-directional interchanges:

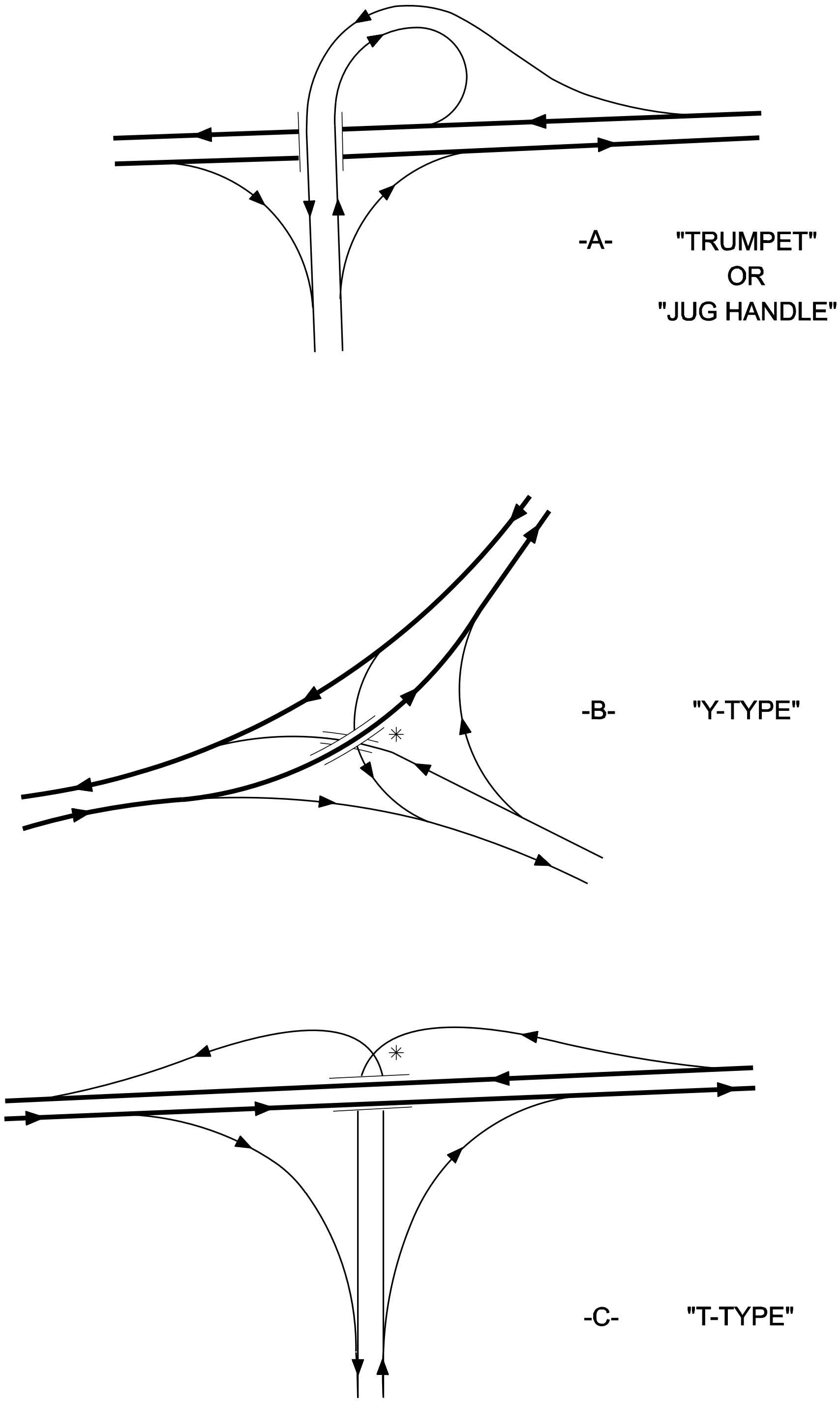
1. Fully Directional Interchange — An interchange where all left-turn movements are provided by directional ramps; see Figure 29.2H.

2. Semi-Directional Interchange — An interchange where one or more left-turn movements are provided by semi-directional ramps, even if the minor left-turn movements are accommodated by loops; see Figure 29.2I.

3. Directional Ramp — A ramp that does not deviate greatly from the intended direction of travel; see Figure 29.2H.

4. Semi-Directional Ramp — A ramp that is indirect in alignment, yet more direct than loops; see Figure 29.2I.

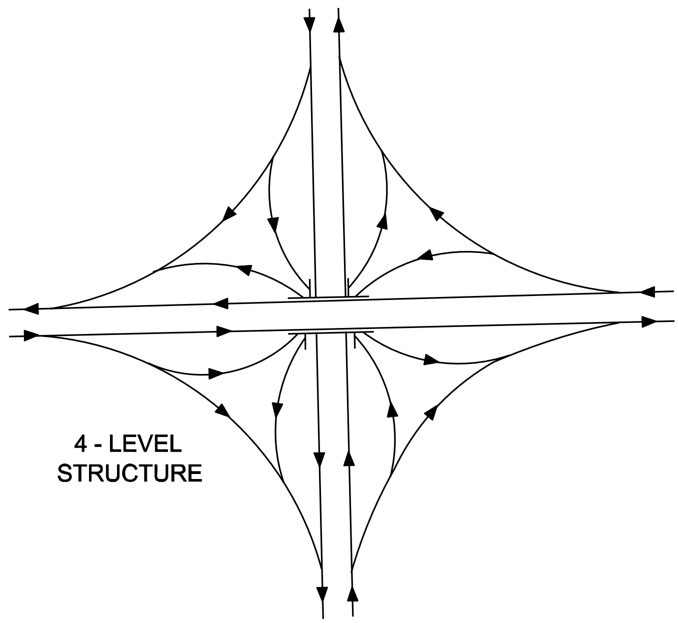
Directional or semi-directional ramps are used for heavy left-turn movements, to reduce travel distance, to increase speed and capacity and to eliminate weaving. These types of connections allow an interchange to operate at a better level of service than is possible with loops. Left-hand exits and entrances may violate driver expectancy and, therefore, should be avoided.



✳*Incorporating an at-grade intersection is an option.*

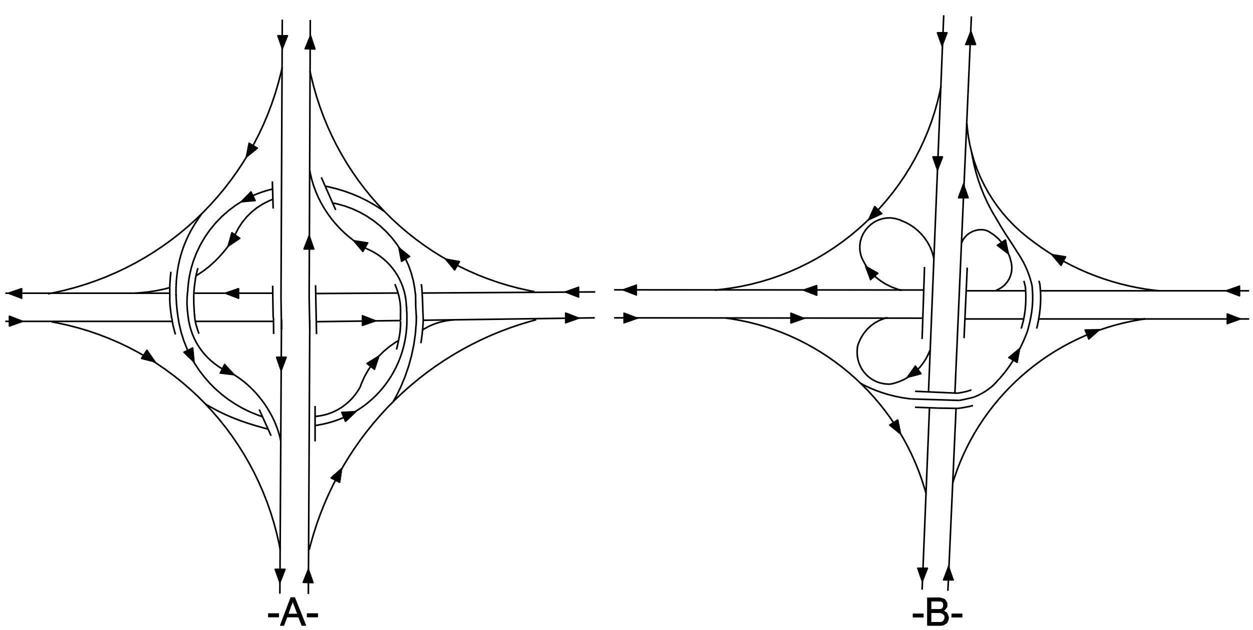
THREE-LEG INTERCHANGES

Figure 29.2G



FULLY DIRECTIONAL INTERCHANGE

Figure 29.2H



SEMI-DIRECTIONAL INTERCHANGES

Figure 29.2I

Directional or semi-directional interchanges are most often provided in urban areas at freeway-to-freeway or freeway-to-arterial intersections. They require less right-of-way than cloverleaf interchanges. A fully directional interchange provides the highest possible capacity and level of service, but it is extremely costly to build because of the multiple-level structure required.

No uniform design procedures can be established for directional and semi-directional ramps at interchanges. Because motorists perceive that higher operating speeds are practical on directional and semi-directional roadways, the alignment of these facilities should be as free flowing as practical.

29.2.2 Selection

Section 29.2.1 presents the typical interchange types that may be used at a given site. For each site, evaluate several interchange types for potential application considering:

1. compatibility with the surrounding highway system and the functional classification of the intersecting highway;
2. route continuity and uniformity with adjacent interchanges;
3. level-of-service for each interchange element (e.g., freeway/ramp junction, ramp proper, ramp/crossroad intersection);
4. operational characteristics (single versus double exits, weaving, signing);
5. road-user impacts (travel distance and time, safety, convenience and comfort);
6. driver expectancy;
7. topography;
8. geometric design;
9. construction and maintenance costs;
10. potential for stage construction;
11. right-of-way impacts and availability;
12. environmental impacts; and
13. potential growth of surrounding area.

In addition, the following will also 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 full access-controlled facilities, the minimum design will be a full cloverleaf interchange. Where traffic volumes are significant, a fully or semi-directional interchange may be the most appropriate interchange type.

3. Movements. All interchanges must 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. 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.

5. 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 usually eliminates the use of a full cloverleaf. In highly restricted locations, the use of a compressed urban interchange or single-point urban interchange may be the only practical option.

    b. Spacing. Closely spaced interchanges may be influenced directly by the preceding or following interchange to the extent 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 will typically 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 urban interchange or compressed diamond will require less right-of-way than a partial cloverleaf, thereby reducing the need to acquire additional right-of-way.

29.3 TRAFFIC OPERATIONAL FACTORS

29.3.1 Basic Number of Lanes

The basic number of lanes is the minimum number of through lanes designated and maintained over a significant length of a route based on the overall operational needs of that highway segment. The basic 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 traffic volume between the exit and entrance drops significantly. Likewise, do not drop a basic lane between closely spaced interchanges simply because the estimated traffic volume in that short section of highway does not warrant the higher number of lanes. Lane drops should only occur where there is a general lowering of the traffic volumes on the overall freeway route.

29.3.2 Lane Balance

Lane balance refers to certain principles that apply at freeway exits and entrances:

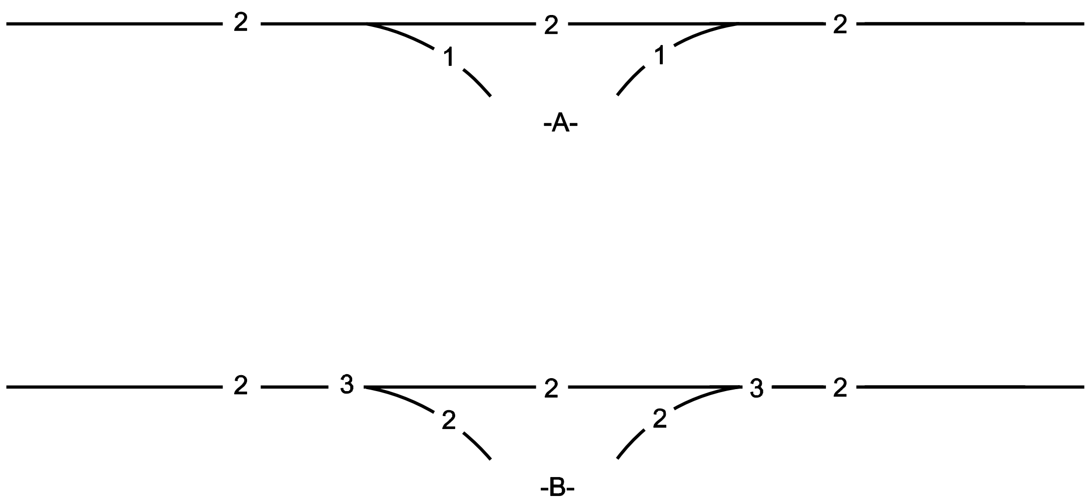
1. Exits. At exits, the number of approach lanes on the highway should equal the sum of the number of mainline lanes beyond the exit plus the number of exiting lanes minus one. An exception to this principle would be at cloverleaf loop ramp exit that follows a loop ramp entrance or at exits between closely spaced interchanges.

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.

For example, dropping two lanes at a 2-lane exit ramp violates the principle of lane balance. One lane should provide the option of remaining on the freeway. Lane balance also prohibits immediately merging both lanes of a 2-lane entrance ramp into a highway mainline without the addition of at least one additional lane beyond the entrance ramp. Figure 29.3A illustrates how to coordinate lane balance and the basic number of lanes at an interchange. Figure 29.3A also illustrates how to achieve lane balance at the merging and diverging points of branch connections.

29.3.3 Route Continuity

All highways with interchanges are designated by route number. The major route should flow continuously through the interchange. The driver should not be required to



COORDINATION OF LANE BALANCE AND BASIC NUMBER OF LANES

Figure 29.3A

change lanes nor to exit in order to remain on the major route. Route continuity 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.

29.3.4 Signing and Marking

Proper interchange operations depend partially 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 impact the minimum acceptable spacing between adjacent interchanges.

29.3.5 Uniformity

Interchange patterns should be uniform from one interchange to another. All ramps should exit and enter on the right. Dissimilar arrangements between interchanges can cause confusion resulting in undesirable lane changes, reduced speeds, etc., especially in urban areas where interchanges are closely spaced.

29.3.6 Distance Between Successive Freeway/Ramp Junctions

Especially in urban areas, successive freeway/ramp junctions frequently may need to be placed relatively close to each other. The distance between the junctions should provide for vehicular maneuvering, signing and capacity. Figure 29.3B provides guidelines for recommended distances for spacings of various freeway/ramp junctions. The ramp-pair combinations are entrance followed by entrance (EN-EN), exit followed by exit (EX-EX), exit followed by entrance (EX-EN) and entrance followed by exit (EN-EX). The criteria in Figure 29.3B are appropriate for the initial planning stages of interchange location. The final spacing between freeway/ramp junctions will be based on the level-of-service criteria and on the detailed capacity methodology in the Highway Capacity Manual.

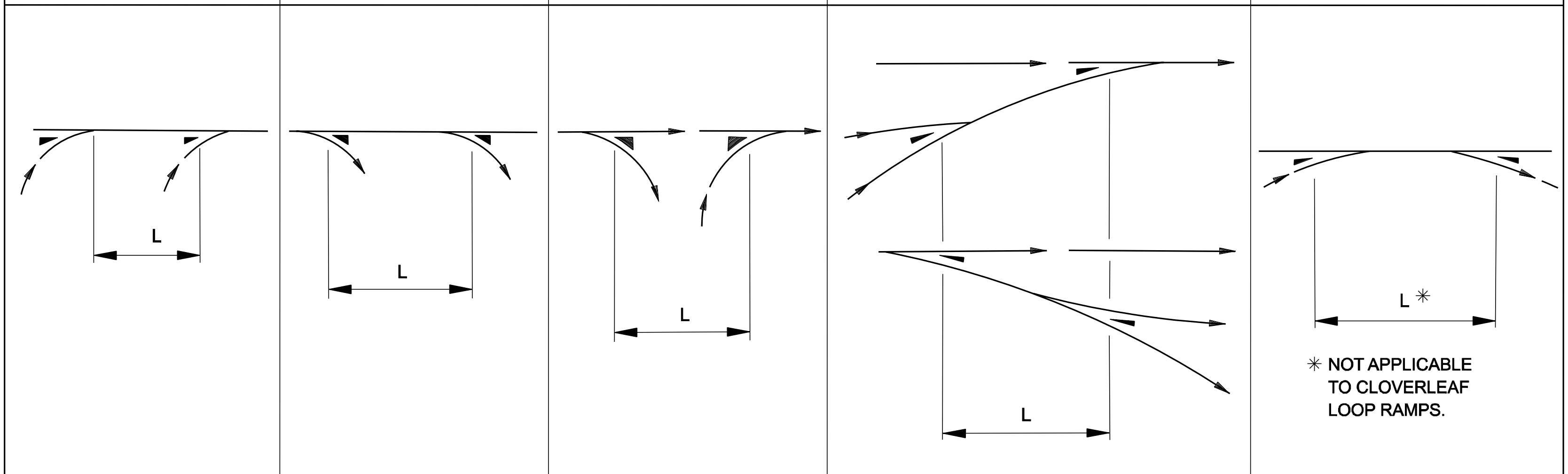
Where the distance between successive entrance and exit terminals is less than 1500 ft (450 m), connect the terminals with an auxiliary lane; see Section 29.3.7.

29.3.7 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. Provide an auxiliary lane where the distance between the end of the entrance terminal and the beginning of an exit terminal is less 1500 ft (450 m). An auxiliary lane may be dropped at an exit if properly signed and designed. The following apply to the use of an auxiliary lane within or near interchanges:

1. Within Interchange. Figure 29.3C provides the basic schematics of alternative designs for adding and dropping auxiliary lanes within an interchange. The selected design will depend upon traffic volumes for the exiting, entering and through movements.

2. Between Interchanges. Where interchanges are closely spaced and an auxiliary lane is warranted at an entrance or exit, the designer should consider connecting the lane to the exit of the downstream interchange or entrance of the upstream interchange.



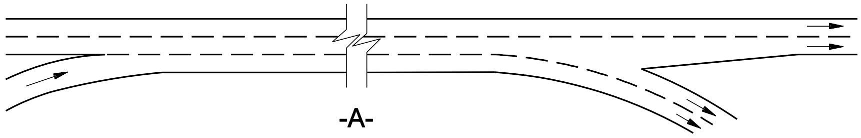
EN-EN	EX-EX		EX-EN		DIRECTIONAL RAMPS		EN-EX (WEAVING)			
FULL FREEWAY CDR OR FDR	FULL FREEWAY	CDR OR FDR	FULL FREEWAY	CDR OR FDR	SYSTEM INTERCHANGE	SERVICE INTERCHANGE	SYSTEM TO SERVICE INTERCHANGE		SERVICE TO SERVICE INTERCHANGE	
							FULL FWY.	CDR OR FDR	FULL FWY.	CDR OR FDR
MINIMUM LENGTHS (L) MEASURED BETWEEN SUCCESSIVE RAMP TERMINALS										
300 ft (100 m)	1000 ft (300 m)	800 ft (240 m)	500 ft (150 m)	400 ft (120 m)	800 ft (240 m)	600 ft (180 m)	2000 ft (600 m)	1600 ft (480 m)	1600 ft (480 m)	1000 ft (300 m)

FDR - FREEWAY DISTRIBUTOR ROAD   CDR - COLLECTOR-DISTRIBUTOR ROAD   EN - ENTRANCE   EX - EXIT

The recommendations are based on operational experience and need for flexibility and adequate signing. They should be checked according to the procedure outlined in the Highway Capacity Manual and Chapter Thirty. The larger of the values is suggested for use. Also, a procedure for measuring the length of the weaving section is given in Chapter 24 of the Highway Capacity Manual.

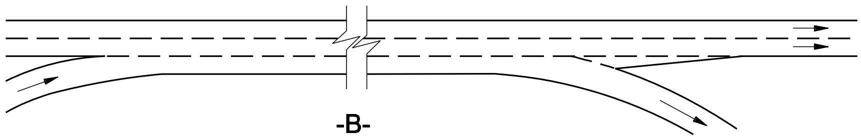
RECOMMENDED MINIMUM RAMP TERMINAL SPACING

Figure 29.3B



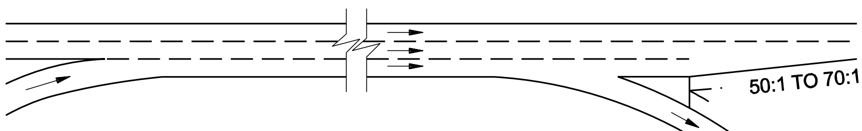
-A-

AUXILIARY LANE DROPPED ON EXIT RAMP



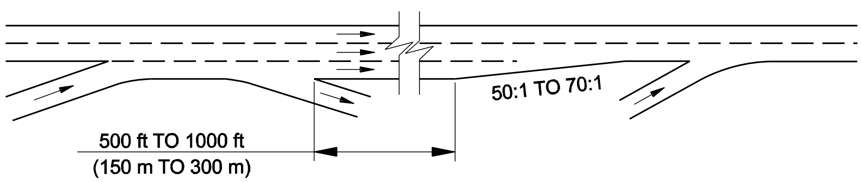
-B-

AUXILIARY LANE BETWEEN CLOVERLEAF LOOPS OR CLOSELY SPACED INTERCHANGES DROPPED ON SINGLE LANE



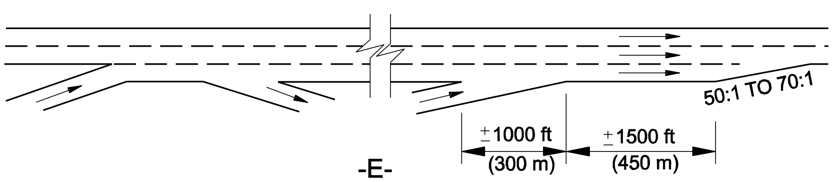
-C-

AUXILIARY LANE DROPPED AT PHYSICAL NOSE



-D-

AUXILIARY LANE DROPPED WITHIN AN INTERCHANGE



-E-

AUXILIARY LANE DROPPED BEYOND AN INTERCHANGE

AUXILIARY LANE DROPS

Figure 29.3C

Design details for exits and entrances are provided in Section 29.5. Auxiliary lane drops beyond the interchange may be merged approximately 2500 ft (750 m) beyond the influence of the last interchange. The design details for lane drops are provided in Section 29.4.5.

29.3.8 Operational/Safety Considerations

Operations and safety are important considerations in the selection and design of an interchange. The following summarizes several major safety considerations:

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

    a. Sight Distance. Where practical, sight distances considerably higher than minimum stopping sight distance (SSD) should be provided to the freeway exit; see Chapter Twenty-four for SSD values. Desirably, use the pavement surface for the height of object (0 ft (0.0 mm)). However, a 2.0 ft (600 mm) height of object is acceptable.

    b. Vertical Alignment. Ramps should depart from the mainline where there will be no vertical curvature to restrict visibility along the ramp. Avoid locating ramps where they drop out of sight.

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

    d. Signing. Provide proper advance signing to the exit to allow all necessary lane changes prior to the exit.

    e. 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; see Section 29.5.1.

2. Entrance Ramps. Provide an acceleration distance of sufficient length to allow a merging vehicle to attain an appropriate speed for merging and to find a sufficient gap in the mainline traffic stream. Where entrance ramps enter the mainline on an upgrade, lengthen the acceleration distance or provide an auxiliary lane to allow entering vehicles to reach an appropriate merging speed; see Section 29.5.2.

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

a. Avoid left-hand terminals. Drivers expect single-lane exits and entrances to be located on the right side of the freeway.

b. Do not mix operational patterns between interchanges or interchange types.

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

4. Fixed-Objects. Because of traffic operations at interchanges, many fixed objects may be located within interchanges (e.g., signs at exit gores, bridge piers, guard rail terminals). These items should be removed, where practical, made breakaway or shielded with barriers or crash cushions. See Chapter Fourteen of the Montana Road Design Manual for Department criteria on roadside safety features.

5. Ramp/Crossroad Terminals. Ensure that the ramp/crossroad terminal has sufficient capacity so that the queuing traffic at the crossroad intersection does not interfere with the freeway or exit ramp operations.

6. Wrong-Way Entrances. In almost all cases, wrong-way maneuvers originate at interchanges. Some simply cannot be avoided, but many result from driver confusion due to poor visibility, confusing ramp arrangement or inadequate signing. Design the interchange to minimize wrong-way possibilities.

7. Weaving. Areas of vehicular weaving may create a high demand on driver skills and attentiveness. Where practical, design the interchange without weaving areas by changing the sequence of ramps, by increasing the spacing between ramps or by using collector-distributor roads.

8. Pedestrians and Bicyclists. Make all crosswalks perpendicular to ramps to reduce the crossing distance. Use appropriate signing and pavement markings to increase the awareness of pedestrians and bicyclists.

29.3.9 Capacity and Level-of-Service

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

1. basic freeway section where interchanges are not present,
2. interchange ramp terminals,
3. weaving areas,
4. ramp proper,

5. ramp merge,
6. ramp diverge, and
7. ramp/crossing road intersection.

The basic capacity reference is the Highway Capacity Manual (HCM). The HCM provides the analytical tools to analyze the level of service for each of these elements. The design year for the interchange and crossroad will typically be the same as that for the freeway (i.e., 20 years).

The interchange should operate at an acceptable level-of-service. The values presented in Chapter Twelve of the Montana Road Design Manual for 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 interchange 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/crossing road intersection in urban areas should not impair the operation of the freeway mainline. This will likely involve a consideration of the operational characteristics on the minor road for some distance in either direction from the interchange. Chapter Thirty provides additional guidance for highway capacity analysis.

29.3.10 Testing 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. The designer should review the plans for areas of possible confusion, proper signing and ease of operation and to determine if sufficient weaving distances and sight distances are available. Review both day and nighttime operations. The designer should consider the peak-hour volumes, number of traffic lanes, etc., to determine the type of traffic the driver will encounter.

29.4 GENERAL DESIGN CONSIDERATIONS

29.4.1 Grade Separation

Section 29.1.4 discusses whether or not to provide an interchange or grade-separated structure. The following discusses how to design a grade separation:

1. Over versus Under. The decision on whether the freeway should go over or under the crossroad is normally dictated by topography and cost. If the topography does not favor one profile over the other, use the following guidelines to determine which highway should cross over the other:

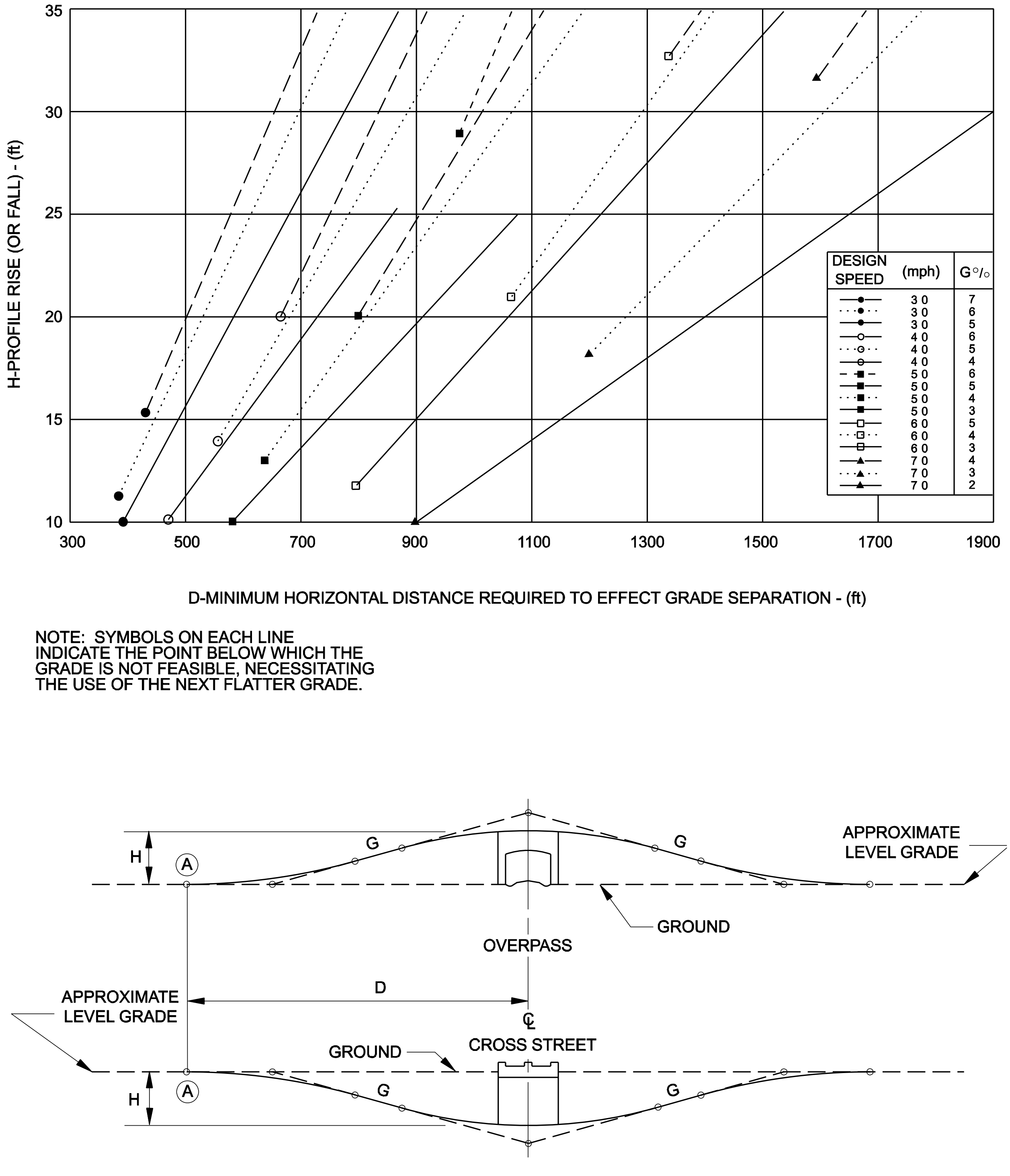
    a. Cost Effectiveness. Evaluate which alternative will be more cost effective to construct. Some elements to consider include the amount of embankment and excavation required, span lengths, angle of skew, gradients, sight distances, alignment, vertical clearances, constructibility, traffic control, right-of-way, drainage and soils conditions.

    b. Gradients. One benefit of the crossroad passing over the freeway is that this may improve the ramp gradients. As drivers exit the freeway, they will normally tend to decelerate going up an exit ramp and accelerate going down an entrance ramp.

    c. Classification. Select the alternative that provides the highest design level for the major road. Typically, the crossroad has a lower design speed and, therefore, it typically can be designed with steeper gradients, lesser widths, reduced vertical clearance requirements, steeper side slopes, etc.

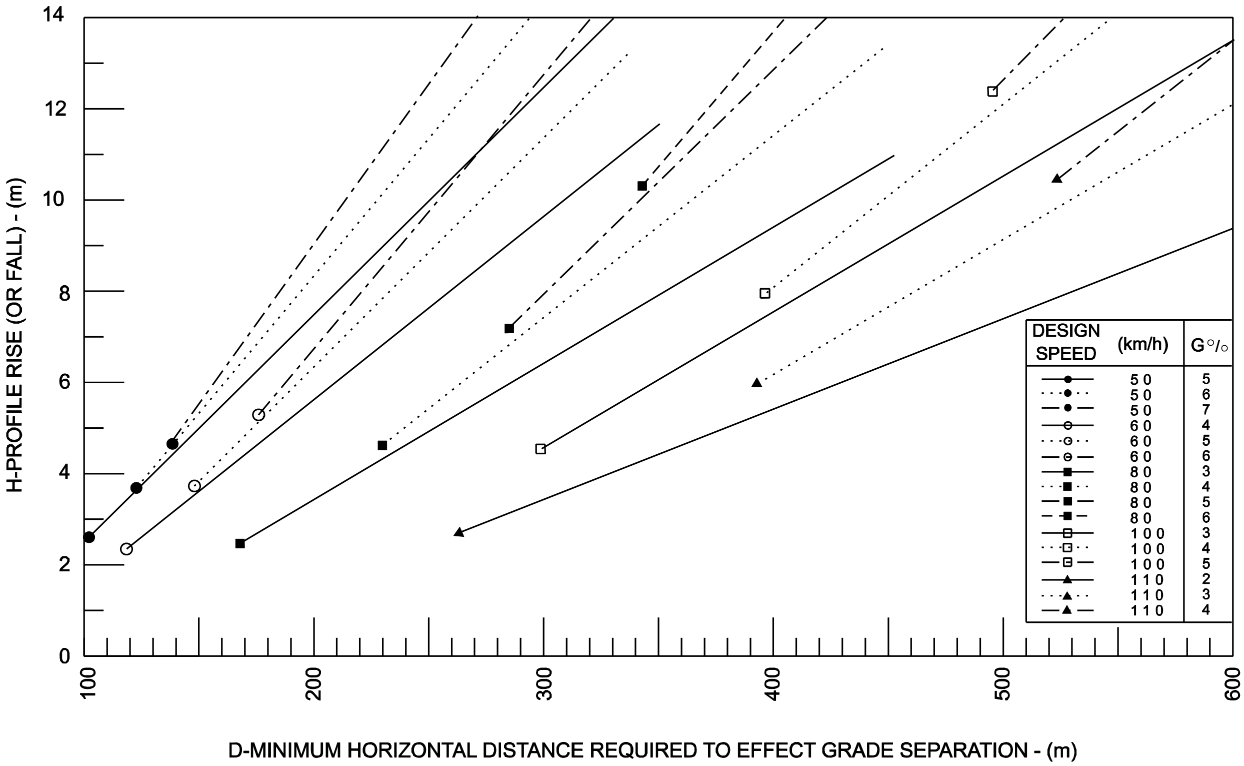
    d. Future Crossings. If any crossings and/or structures are planned for a future date, the mainline should pass under these future crossings. Overpasses are easier to install and will be less disruptive to the major road when they are constructed in the future.

2. Horizontal Distance. The distance required for adequate design of a grade separation depends on the design speed, the roadway gradient and the amount of rise or fall necessary to produce the separation. Figure 29.4A can be used in the preliminary design phase to quickly determine whether a grade separation is feasible for a given set of conditions, what gradients may be involved and what profile adjustments may be necessary on the crossroad. Also, carefully study sight distance requirements because these will often dictate the necessary horizontal distance. When using Figure 29.4A, consider the following:

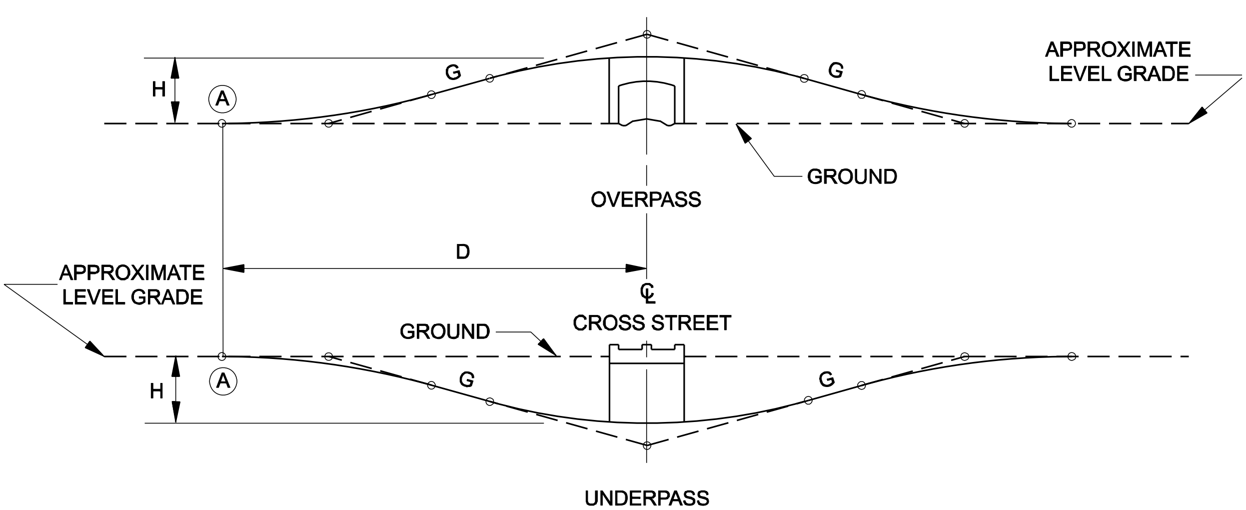


HORIZONTAL DISTANCE FOR GRADE SEPARATION
(US Customary)

Figure 29.4A



NOTE: SYMBOLS ON EACH LINE INDICATE THE POINT BELOW WHICH THE GRADE IS NOT FEASIBLE, NECESSITATING THE USE OF THE NEXT FLATTER GRADE.



HORIZONTAL DISTANCE FOR GRADE SEPARATION
(Metric)

Figure 29.4A

a. Minimum Horizontal Distances. The plotted lines in Figure 29.4A are derived assuming the same approach gradient on each side of the structure. However, values of "D" from the figure also are applicable to combinations of unequal gradients. Distance "D" is equal to the length of the initial vertical curve, plus one-half the central vertical curve, plus the length of tangent between the curves. Lengths of vertical curves are based on the minimum stopping sight distance. Longer vertical curves are desirable from an aesthetic and safety perspective. However, longer curve lengths are more costly due to increased earthwork.

b. Maximum Gradient. The lower terminal point of the gradient lines in Figure 29.4A, marked by a small symbol, indicates the distance where the tangent between the curves is zero and below which a design for the given grade is not feasible (i.e., a profile condition where the minimum central and end curves for the gradient would overlap).

c. Restricted Gradients. For the usual profile rise or fall required for a grade separation ("H" of 25 ft (7.5 m) or less), do not use gradients greater than 3% for a design speed of 70 mph (110 km/h), 4% for 60 mph (100 km/h), 5% for 50 mph (80 km/h), and 6% for 35 mph (60 km/h). For values of "H" less than 25 ft (7.5 m), use flatter gradients.

d. Relationship. For a given "H" and design speed, distance "D" is only shortened a negligible amount by increasing the gradient. However, the distance "D" varies to a greater extent for a given "H" and "G" with respect to the design speed.

e. Elevation. Considering the vertical clearance and structural depth, an elevation distance of "H" is typically between 20 ft (6.0 m) and 22 ft (6.6 m) for the grade separation of two highways. "H" is typically the same for a freeway under a railroad. For a railroad facility under a freeway, "H" is typically 28 ft (8.5 m).

29.4.2 Underpass Width

The approach cross section, desirably including clear zones, should be carried through the underpass. Including the clear zone allows for possible expansion in the future with minimal disruption to the overhead structure. In addition, wider underpasses also provide greater sight distance for at-grade ramp terminals near the structure.

29.4.3 Collector-Distributor Roads

In general, interchanges that are designed with single exits are superior to those with two exits, especially if one of the exits is a loop ramp or the second exit is a loop ramp preceded by a loop entrance ramp. Whether used in conjunction with a full cloverleaf or with a partial cloverleaf interchange, the single-exit design may improve the operational efficiency of the entire interchange.

Collector-distributor (C-D) roads use the single-exit approach to improve the interchange operational characteristics. C-D roads will:

1. remove weaving maneuvers from the mainline and transfer them to the lower speed C-D roads,
2. provide high-speed single exits and entrances from and onto the mainline,
3. satisfy driver expectancy by placing the exit in advance of the separation structure,
4. simplify signing and the driver decision-making process, and
5. provide uniformity of exit patterns.

C-D roads are most often warranted where traffic volumes are so high that the interchange without them cannot operate at an acceptable level-of-service, especially in weaving sections. They are particularly advantageous at full cloverleaf interchanges where the weaving between the ramp/mainline traffic can be very difficult. Figure 29.2E illustrates a schematic of a C-D within a full cloverleaf interchange.

C-D roads may be one or two lanes, depending upon the traffic volumes and weaving conditions. Lane balance should be maintained at the exit and entrance points of the C-D road. The design speed of a C-D road usually ranges from 40 mph to 50 mph (60 km/h to 80 km/h); however, it should desirably be within 10 mph (20 km/h) of the mainline design speed. The separation between the C-D road and mainline should be as wide as practical but not less than that required to provide the applicable shoulder widths and a longitudinal median barrier between the two.

29.4.4 Frontage Roads

29.4.4.1 General

Frontage roads serve numerous functions, depending on the type of facility served and the character of the surrounding area. They may be used to control access to the facility, to function as a street serving adjoining property and to maintain circulation of traffic on each side of the main highway. Frontage roads segregate local traffic from the higher speed through traffic and serve driveways of residences and commercial establishments along the highway. Connections between the main highway and frontage roads, usually provided at crossroads, furnish access between through roads and adjacent property. Thus, the through character of the highway is preserved and is unaffected by subsequent development along the roadsides. Chapter Eighteen of the Montana Road Design Manual presents design details for frontage roads. Ensure an operable distance is maintained between the frontage road and the ramp terminal.

29.4.4.2 Freeway Connections

Connections between the freeway and the frontage road are an important design element. In general, access is provided from the crossroad beyond the interchange, see Figure 29.2B. Slip ramps from one-way frontage roads and freeways are also acceptable. However, slip ramps from a freeway to two-way frontage roads are undesirable because they tend to induce wrong-way entry onto the freeway and may cause crashes at the intersection of the ramp and frontage road. Therefore, on freeways and other arterials with high operating speeds and two-way frontage roads, the access to the freeway must be provided at interchanges. Figure 29.4B illustrates details for the ramp/frontage road design with one-way frontage roads.

The design in Figure 29.4B may only be used in restricted urban areas. The critical design element is the distance "A" between the ramp/frontage road merge and the crossing road. This distance must be sufficient to allow traffic weaving, vehicular deceleration and stopping, and vehicular storage to avoid interference with the merge point. Figure 29.4B also presents general guidelines that may be used to estimate this distance during the preliminary design phase. A number of assumptions have been made including weaving volume, operating speeds and intersection queue distance. Therefore, a detailed analysis will be necessary to firmly establish the needed distance to properly accommodate vehicular operation. Additional information can be found in a Transportation Research Record 682 paper entitled, "Distance Requirements for Frontage-Road Ramps to Cross Streets: Urban Freeway Design."

Distance "B" in Figure 29.4B is determined on a case-by-case basis. It should be determined based on the number of frontage road lanes and the intersection design. This distance is typically determined by the weaving distance from the intersection to ramp entrance. For capacity analysis of the weaving section, see the Highway Capacity Manual. Under some circumstances this distance may be 0.0 ft (m).

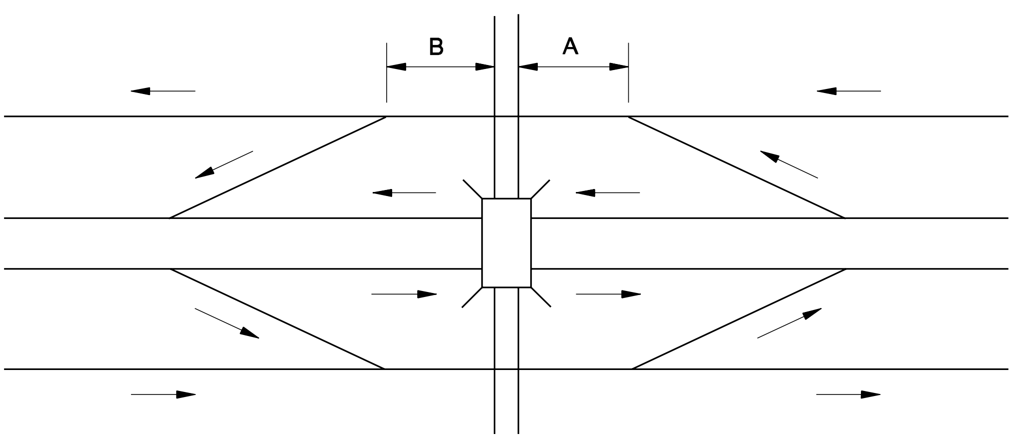
Frontage Road Volume (vph)[1]	Exit Ramp Volume (vph)[2]	"A" (ft)		
		Typical Minimum	Typical Desirable	Special Conditions
200	140	380	500	260
400	275	460	560	360
600	410	500	630	400
800	550	540	690	430
1000	690	590	760	450
1200	830	640	870	480
1400	960	690	970	500
1600	1100	770	1070	530
1800	1240	860	1180	550
2000	1380	970	1300	580

[1] *Total frontage road and exit ramp volume between merge to intersection with minor road.*

[2] *Assumed to be 69% of total volume in first column.*

Note: Table values are acceptable for planning purposes; final dimensions will be based on a detailed operational analysis. This design may be used where necessary in restricted urban areas.

Distance B is typically determined by the weaving distance from the intersection to the ramp entrance, see Section 29.4.4.



RAMP/CONTINUOUS FRONTAGE ROAD INTERSECTION
(US Customary)

Figure 29.4B

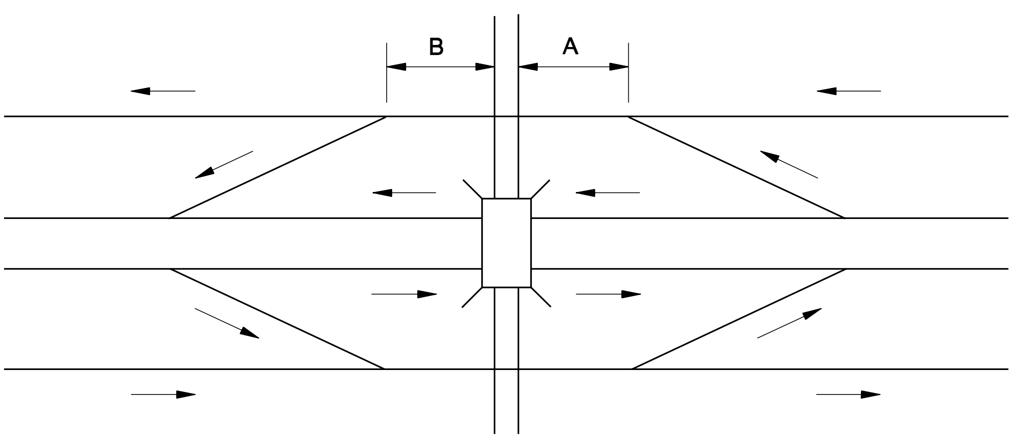
Frontage Road Volume (vph)[1]	Exit Ramp Volume (vph)[2]	"A" (m)		
		Typical Minimum	Typical Desirable	Special Conditions
200	140	115	150	80
400	275	140	170	110
600	410	150	190	120
800	550	165	210	130
1000	690	180	230	140
1200	830	195	265	145
1400	960	210	295	150
1600	1100	235	325	160
1800	1240	260	360	170
2000	1380	295	395	180

(1) *Total frontage road and exit ramp volume between merge to intersection with minor road.*

(2) *Assumed to be 69% of total volume in first column.*

Note: Table values are acceptable for planning purposes; final dimensions will be based on a detailed operational analysis. This design may be used where necessary in restricted urban areas.

Distance B is typically determined by the weaving distance from the intersection to the ramp entrance, see Section 29.4.4.



RAMP/CONTINUOUS FRONTAGE ROAD INTERSECTION
(Metric)

Figure 29.4B

The following summarizes the available options for coordinating the design of the interchange ramps, frontage roads and crossing roads:

1. Slip Ramps. Slip ramps may be used to connect the freeway with one-way frontage roads before (or after) the intersection with the crossing road.

2. Separate Intersections. Separate ramp/crossing road and frontage road/crossing road intersections may be accomplished by curving the frontage road away from the ramp and intersecting the frontage road with the crossing road outside the ramp limits of full-access control. Figure 29.2B provides an illustration of this separation. This treatment allows the two intersections to operate independently, and it eliminates the operational and signing problems of providing the same point of exit and entrance for the frontage road and freeway ramp.

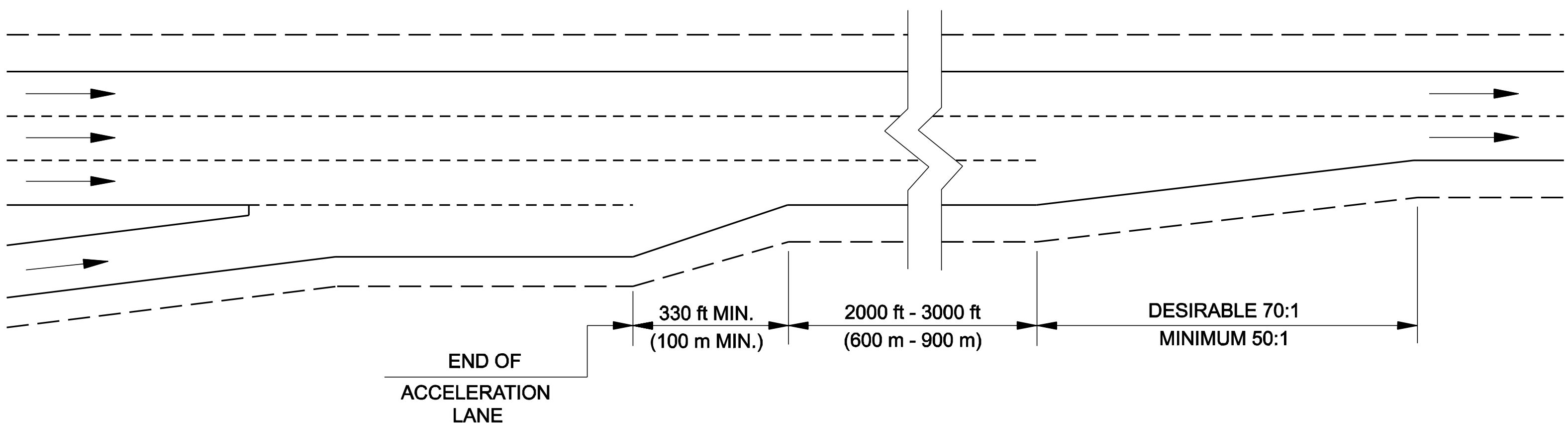
29.4.5 Freeway Lane Drops

Freeway lane drops, where the basic number of lanes is decreased, must be carefully designed. They should normally occur on the freeway mainline away from any other turbulence (e.g., interchange exits and entrances, emergency crossovers). However, it may be advantageous to drop a basic freeway lane at a 2-lane exit. Figure 29.4C illustrates the recommended design of a lane drop beyond an interchange. The following criteria are important:

1. Location. Desirably, the lane drop should occur approximately 2000 ft – 3000 ft (600 m – 900 m) beyond the previous interchange ramp. Under restricted conditions, the MUTCD signing distance is acceptable. The 2000 ft – 3000 ft (600 m – 900 m) allows for adequate signing and driver adjustments from the interchange, but yet is not so far downstream that drivers become accustomed to the number of lanes and are surprised by the lane drop. In addition, do not drop a lane on a horizontal curve or where other signing is required (e.g., an upcoming exit).

   In urban areas, interchanges may be closely spaced for considerable lengths of highway. In these cases, it may be necessary to drop a freeway lane at an exit. Where this is necessary, it is preferable to drop a freeway lane at a 2-lane exit rather than a single-lane exit. As discussed in Section 29.3.1, a lane should not be dropped at an exit unless there is a corresponding decrease in traffic demand for a significant length of freeway (e.g., 10 mi (15 km)).

2. Transition. Desirably, the transition taper length will be 70:1. The minimum taper rate is 50:1; see Figure 29.4C.



FREEWAY LANE DROP
(Typical Schematic)

Figure 29.4C

3. Sight Distance. Sufficient sight distance should be available to any point within the entire lane transition. When determining the sight distance availability, desirably the height of object will be 0.0 ft (mm) (the roadway surface); however, it is acceptable to use 2 ft (600 mm). This criteria would favor, for example, placing a freeway lane drop within a sag vertical curve rather than just beyond a crest.

4. Lane Drop. Right-side freeway lane drops are preferred versus left-side lane drops.

5. Shoulders. Maintain the full-width shoulder through a lane drop. This provides an area to allow a driver who may have missed the signing an opportunity to safely merge with the through traffic.

29.4.6 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 function and 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 too steep to compromise safety and that they can support plantings that 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 also 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 in the plans.

29.5 FREEWAY/RAMP JUNCTIONS

29.5.1 EXIT RAMPS

29.5.1.1 Types of Exit Ramps

There are two basic types of exit freeway/ramp junctions — the parallel design and the taper design. Figures 29.5A and 29.5B illustrate these designs. For most new and reconstructed ramps, it is MDT policy to use the taper design (Figure 29.5A). However, the designer may consider using the parallel design (Figure 29.5B) where:

1. a ramp exit is just beyond a structure and there is insufficient sight distance available to the ramp gore;

2. the need is satisfied for a continuous auxiliary lane (see Section 29.3.7); or

3. 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 operation. It is also easier to design the superelevation transition with a parallel design. The design speed of the departure angle or exit curve must equal the design speed of the roadway being exited.

29.5.1.2 Taper Rates

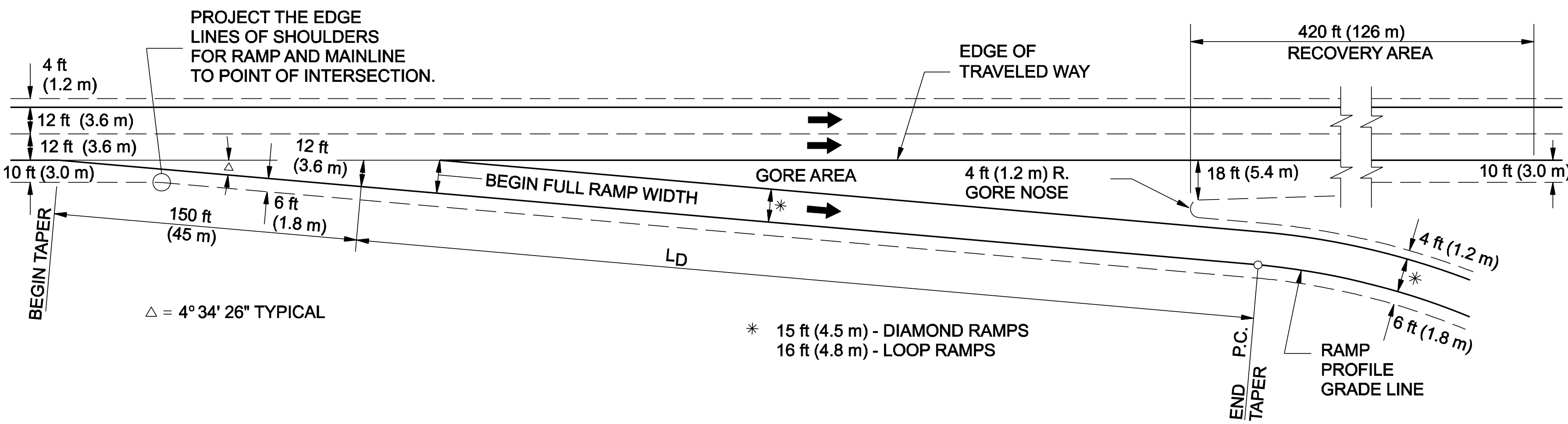
The taper rate applies to the rate at which the ramp diverges from the freeway. The following taper rates apply:

1. Taper Exit Design. The taper angle can vary between 2° and 5°. For the typical MDT ramp design, the divergence angle is 4° 34′ 26″ as illustrated in Figure 29.5A.

2. Parallel Exit Design. The taper rate applies to the beginning of the parallel lane. This distance is typically 215 ft (65 m) as illustrated in Figure 29.5B.

29.5.1.3 Deceleration

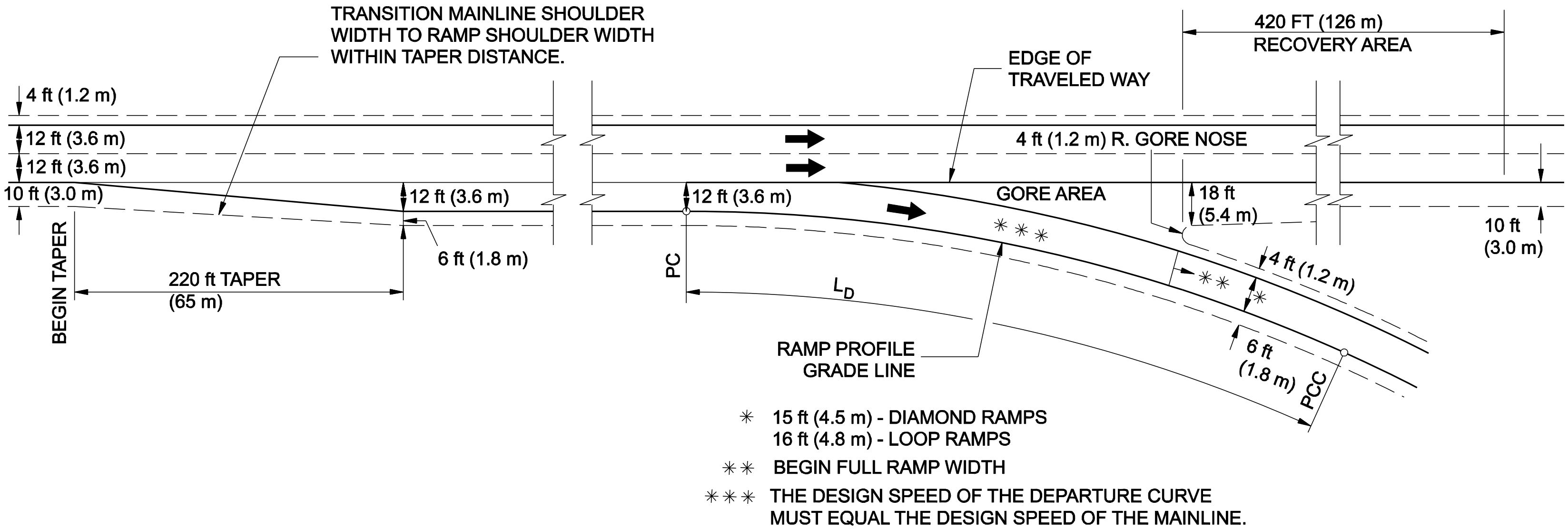
Sufficient deceleration is needed to safely and comfortably allow an exiting vehicle to leave the mainline. There are two types of freeway departures, the taper exit type and the parallel exit type.



Note: L_D is the deceleration distance required for a vehicle to slow down from the mainline design speed to the design speed of first geometric control on the ramp; see Section 29.5.1.3.

TAPER EXIT RAMP

Figure 29.5A



Note: L_D is the deceleration distance required for a vehicle to slow down from the mainline design speed to the design speed of first geometric control on the ramp; see Section 29.5.1.3.

PARALLEL EXIT RAMP

Figure 29.5B

In the taper exit type, 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 ft (3.6 m) wide to the first geometric control.

In the parallel exit type, the departure curve or taper should equal the design speed of the roadway being departed. The deceleration should be planned to be within the departure curve or taper, not in the parallel lane. This is due to the potential for motorist to make late decisions to exit the roadway at the separation point.

Figure 29.5C 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% or more, adjust the deceleration distance according to the criteria in Figure 29.5D. Where there are significant trucks exiting the mainline (e.g., weigh stations, truck stops, rest areas), the designer should consider increasing the deceleration distance according to the criteria in Figure 29.5E.

29.5.1.4 Sight Distance

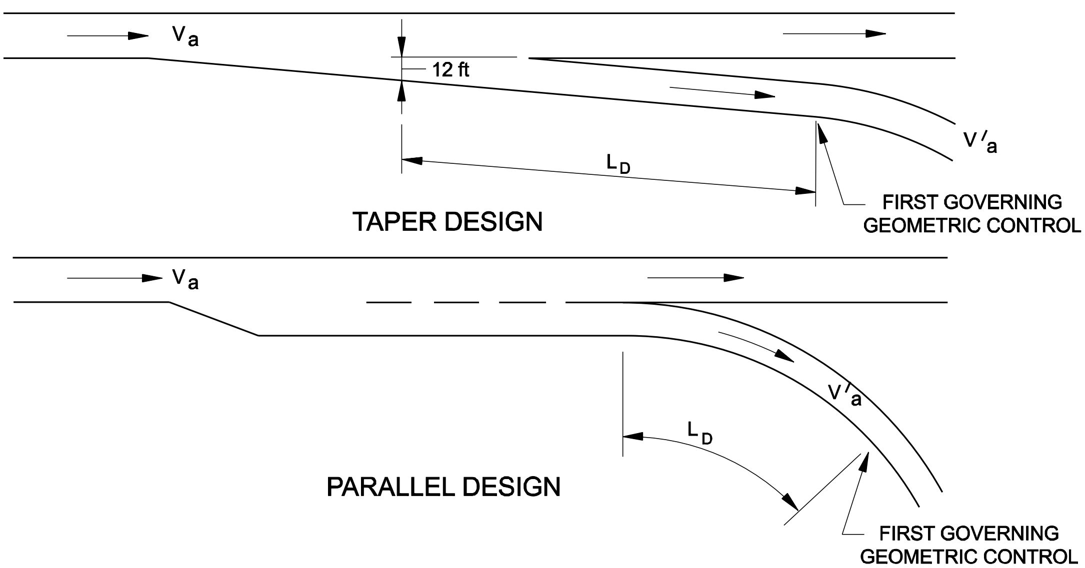
A sight distance of 1180 ft (360 m) should be provided for drivers approaching a freeway exit. The 1180 ft (360 m) distance should be available throughout the freeway/ramp junction (e.g., from the Begin Taper to the Gore Nose, see Figures 29.5A and 29.5B). This sight distance is particularly important 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 ft (mm) (the roadway surface); however, it is acceptable to use 2 ft (600 mm).

29.5.1.5 Superelevation

Superelevation for horizontal curves at the freeway/ramp junction will be developed based on the principles of superelevation for open-roadway conditions, as discussed in Chapter Twenty-five of the Montana Traffic Engineering Manual and Chapter Nine of the Montana Road Design Manual. The following criteria are applicable to superelevation development at exit ramps:

1. Design Speed. As discussed in Section 29.5.1.3, 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. In most cases, this will be a horizontal curve in the vicinity of the exit gore. If the

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



Notes:

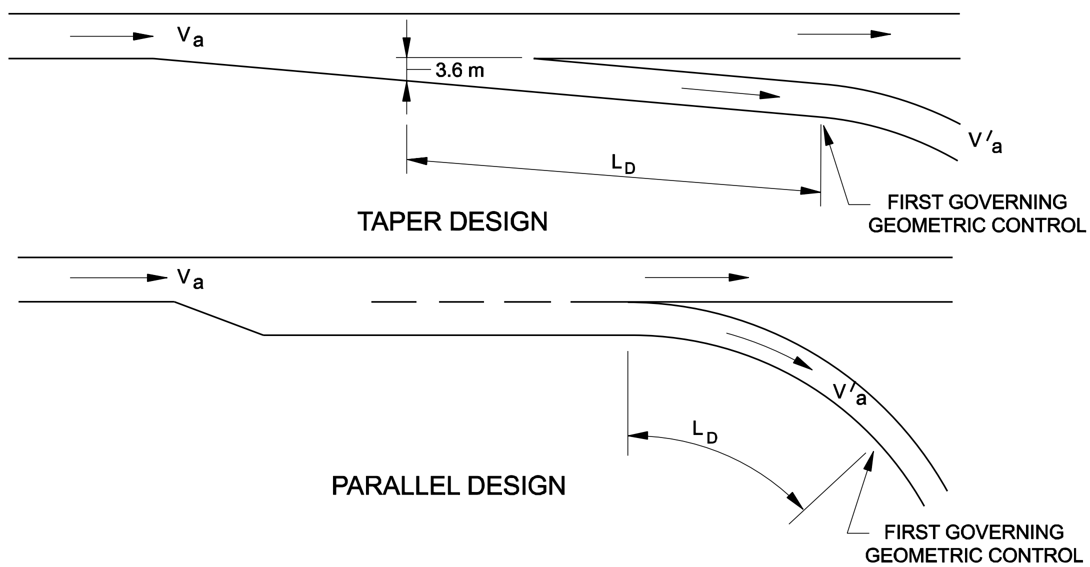
1. *The deceleration lengths are calculated from the distance needed for a passenger car to decelerate from the average running speed of the highway mainline to the average running speed of the first governing geometric control.*

2. *These values are for grades less than 3%. See Figure 29.5D for steeper downgrades.*

LENGTH FOR DECELERATION (PASSENGER CARS)
(US Customary)

Figure 29.5C

Highway Design Speed (km/h) (V)	Actual Speed (km/h) (V_a)	L_D = Deceleration Length (m)							
		For Design Speed of First Governing Geometric Control (km/h) (V′)							
		Stop	20	30	40	50	60	70	80
		For Average Running Speed on Exit Curve (km/h) (V_a′)							
		0	20	28	35	42	51	63	70
50	47	75	70	60	45	-	-	-	-
60	55	95	90	80	65	55	-	-	-
70	63	110	105	95	85	70	55	-	-
80	70	130	125	115	100	90	80	55	-
90	77	145	140	135	120	110	100	75	60
100	85	170	165	155	145	135	120	100	85
110	91	180	180	170	160	150	140	120	105
120	98	200	195	185	175	170	155	140	120



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%. See Figure 29.5D for steeper downgrades.*

LENGTH FOR DECELERATION (PASSENGER CARS)
(Metric)

Figure 29.5C

Direction of Grade	Ratio of Deceleration Length on Grade to Length on Level			
	< 3%	3% $\leq$ G < 5%	5% $\leq$ G < 7%	G $\geq$ 7%
Downgrade	1.0	1.2	1.35	1.5

Notes: 1. Table 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 29.5A and 29.5B.

GRADE ADJUSTMENTS FOR DECELERATION LENGTHS

Figure 29.5D

necessary deceleration distance is available, then the design speed of the horizontal curve may be equal to the design speed of the ramp proper; see Section 29.6.

2. Maximum Superelevation. The typical e_{max} is 8%.

3. Superelevation Rate. Use Figure 25.3A to determine the proper superelevation rate for horizontal curves at freeway/ramp exits. The designer will use the selected design speed and the curve radius to read into the tables to determine "e."

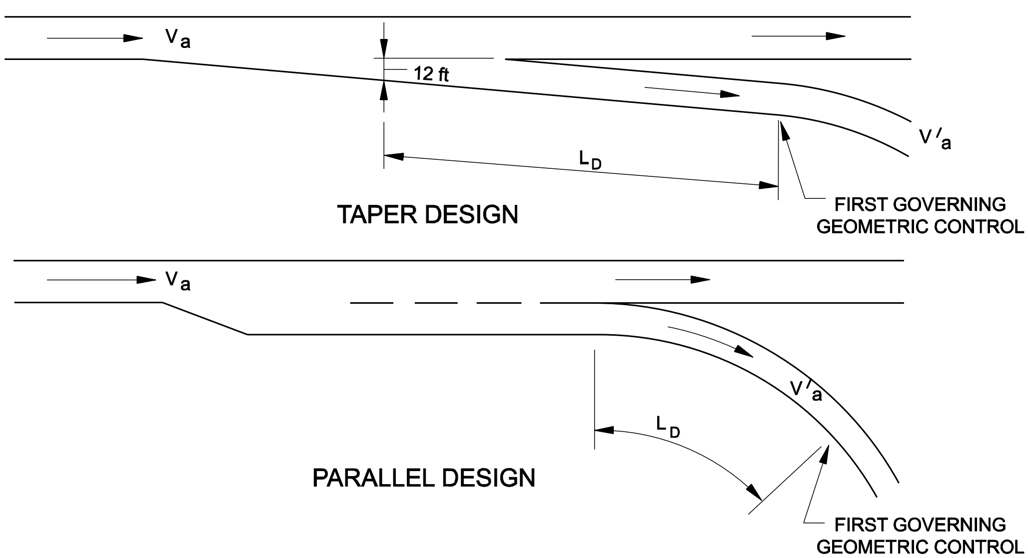
4. Transition Length. The designer must transition the exit ramp cross slope (typically 2%) to the superelevation rate for the horizontal curve. The following applies:

   a. The transition should not begin until the exit ramp has reached a minimum 12 ft (3.6 m) width.

   b. The maximum relative longitudinal gradient should not exceed the criteria in Figure 9.3C in the Montana Road Design Manual. The relative gradient is measured between the outside edge of ramp traveled way and the inside edge of ramp traveled way.

   c. The minimum transition length should be based on the criteria in Figure 9.3D in the Montana Road Design Manual.

Highway Design Speed (mph) (V)	Speed Reached (mph) (V_a)	L_D = Deceleration Length (ft)								
		For Design Speed of First Governing Geometric Control (mph) (V′)								
		Stop	15	20	25	30	35	40	45	50
		For Average Running Speed on Exit Curve (mph) (V_a')								
		0	14	18	22	26	30	36	40	44
30	28	270	230	200	160	—	—	—	—	—
35	32	340	295	265	230	190	—	—	—	—
40	36	415	370	340	305	260	210	—	—	—
45	40	495	455	425	390	345	295	210	—	—
50	44	585	540	510	475	435	385	295	230	—
55	48	685	640	610	570	530	480	390	320	250
60	52	785	740	710	675	635	585	495	425	350
65	55	865	825	795	760	715	665	575	510	435
70	58	955	910	880	845	800	750	665	595	520
75	61	1040	995	970	935	890	840	750	685	610



Notes:

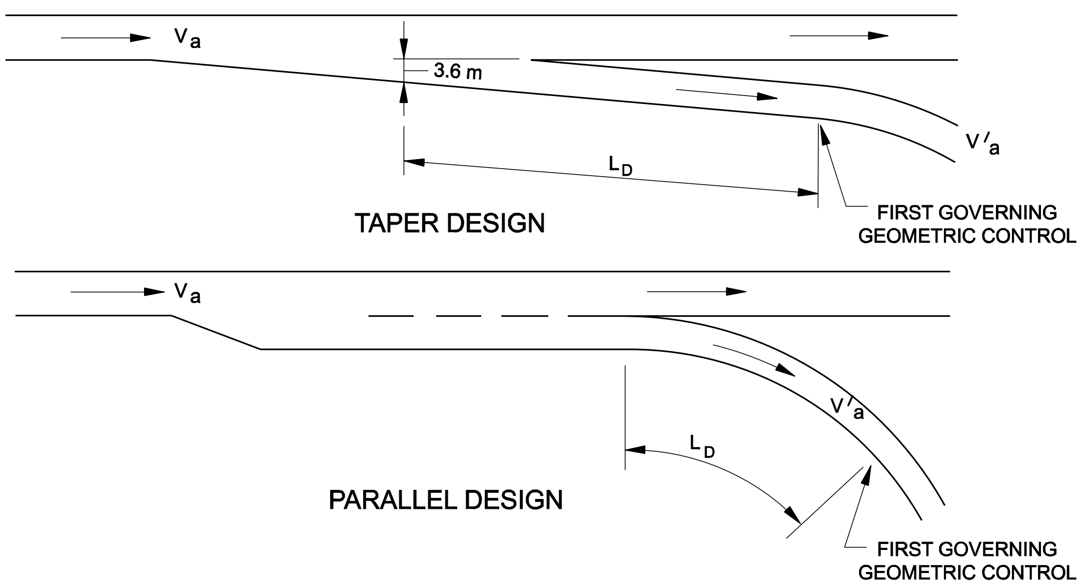
1. *The deceleration lengths are calculated from the distance needed for a 200 lb/hp truck to decelerate from the average running speed of the highway mainline to the average running speed of the first governing geometric control.*

2. *These values are for grades less than 3%. See Figure 29.5D for steeper downgrades.*

LENGTH FOR DECELERATION (200 LB/HP TRUCKS)
(US Customary)

Figure 29.5E

Highway Design Speed (km/h) (V)	Actual Speed (km/h) (V_a)	L_D = Deceleration Length (m)							
		For Design Speed of First Governing Geometric Control (km/h) (V′)							
		Stop	20	30	40	50	60	70	80
		For Average Running Speed on Exit Curve (km/h) (V_a')							
		0	20	28	35	42	51	63	70
50	47	90	80	70	60	—	—	—	—
60	55	120	105	95	85	—	—	—	—
70	63	150	135	125	115	100	—	—	—
80	70	175	165	155	145	130	—	—	—
90	77	210	200	190	175	165	140	105	—
100	85	250	235	225	215	200	180	145	120
110	91	280	270	260	245	235	210	175	150
120	98	320	305	295	285	270	250	215	190



Notes:

1. *The deceleration lengths are calculated from the distance needed for a 120 kg/kW truck 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%. See Figure 29.5D for steeper downgrades.*

LENGTH FOR DECELERATION (120 KG/KW TRUCKS)
(Metric)

Figure 29.5E

d. Approximately 70% of the transition length should be on the tangent and approximately 30% on the curve.

5. Axis of Rotation. The axis of rotation is typically about the inside edge of the ramp traveled way at the freeway/ramp junction and about the outside edge of ramp traveled way on the ramp proper.

* * * * * * * * *

Example 29-5.1

Given:	Highway Design Speed	-	70 mph
	First Exit Curve Design Speed	-	45 mph
	Average Grade	-	5% downgrade

Problem: Determine length of deceleration required.

Solution: Figure 29.5C yields a minimum deceleration length of 390 ft on the level. According to Figure 29.5D, this should be increased by 1.35.

Therefore: L = 390 x 1.35
L = 527 ft

Provide a 527 ft deceleration length from the full width of the exit lane to the PC of the first exit curve.

* * * * * * * * *

29.5.1.6 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 a range of 4% to 5%.

2. From Physical Nose to Gore Nose. The cross slope rollover should not exceed 8%.

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 Item #'s 1 or 2 above, depending on the inlet location.

See Section 29.5.1.8 for nose definitions.

29.5.1.7 Shoulders

The wider right shoulder of the mainline must be transitioned to the narrower shoulder of the ramp (i.e., 10 ft to 6 ft (3.0 m to 1.8 m)). The shoulder width should be transitioned as shown in Figures 29.5A and 29.5B.

29.5.1.8 Gore Area

The gore area is normally considered to be both the paved triangular area between the through lane and the exit ramp, plus the graded area which may extend approximately 350 ft (100 m) downstream beyond the gore nose. The following definitions will apply (see Figure 29.5F):

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.

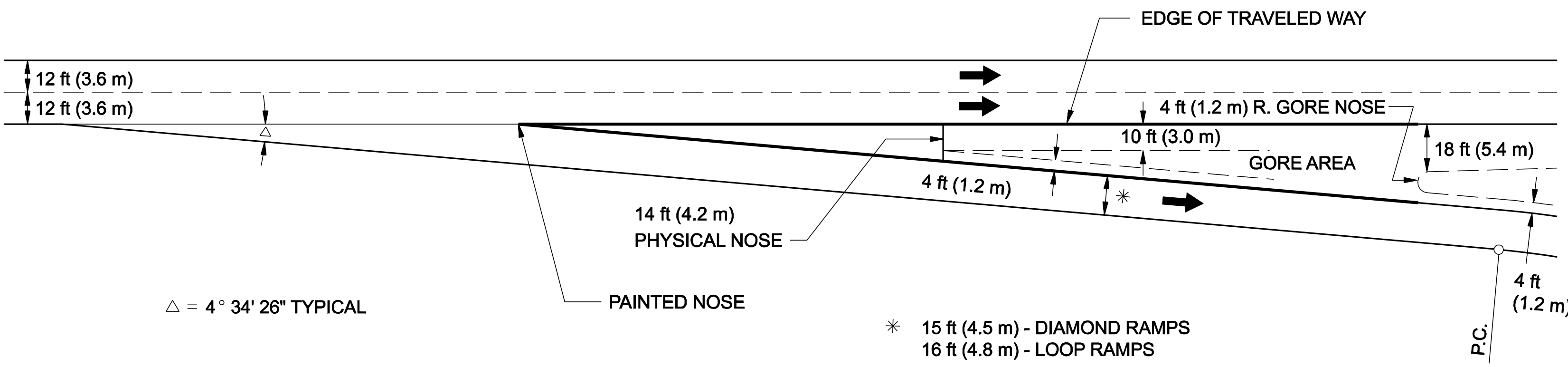
2. Physical Nose. This is the point where the ramp and mainline shoulders converge. As illustrated in Figure 29.5F, the physical nose has a dimensional width of 14 ft (4.2 m).

3. Gore Nose. This is the point where the paved shoulder ends and the sodded area begins as the ramp and mainline diverge from one another. As illustrated in Figure 29.5F, the gore nose is rounded with a 4 ft (1.2 m) radius.

The following should be considered 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 ft (30 m) beyond the gore nose. Any obstacles within 350 ft (100 m) of the gore nose must be made breakaway or shielded by a barrier; see Chapter Six of this Manual and Chapter Fourteen of the Montana Road Design Manual.

2. 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-



GORE AREA CHARACTERISTICS

Figure 29.5F

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 Chapter Six of this Manual and Chapter Fourteen of the Montana Road Design Manual.

3. Cross Slopes. The paved triangular gore 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%) from the painted nose up to the physical nose. Beyond this point, the gore area is depressed with cross slopes of 2% - 4%. See Section 29.5.1.6 for criteria on breaks in cross slopes within the gore area.

4. Traffic Control Devices. Signing in advance of the exit and at the divergence should be according to the MUTCD and Chapter Eighteen. See Chapter Nineteen for the pavement marking details in the triangular area upstream from the gore nose.

5. Recovery Area. Where crash history indicates a problem or where it may be confusing for the exiting or through driver, the designer may consider providing a recovery area for 500 ft to 1000 ft (150 m to 300 m) beyond the gore nose; see Figures 29.5A and 29.5B.

29.5.2 Entrance Ramps

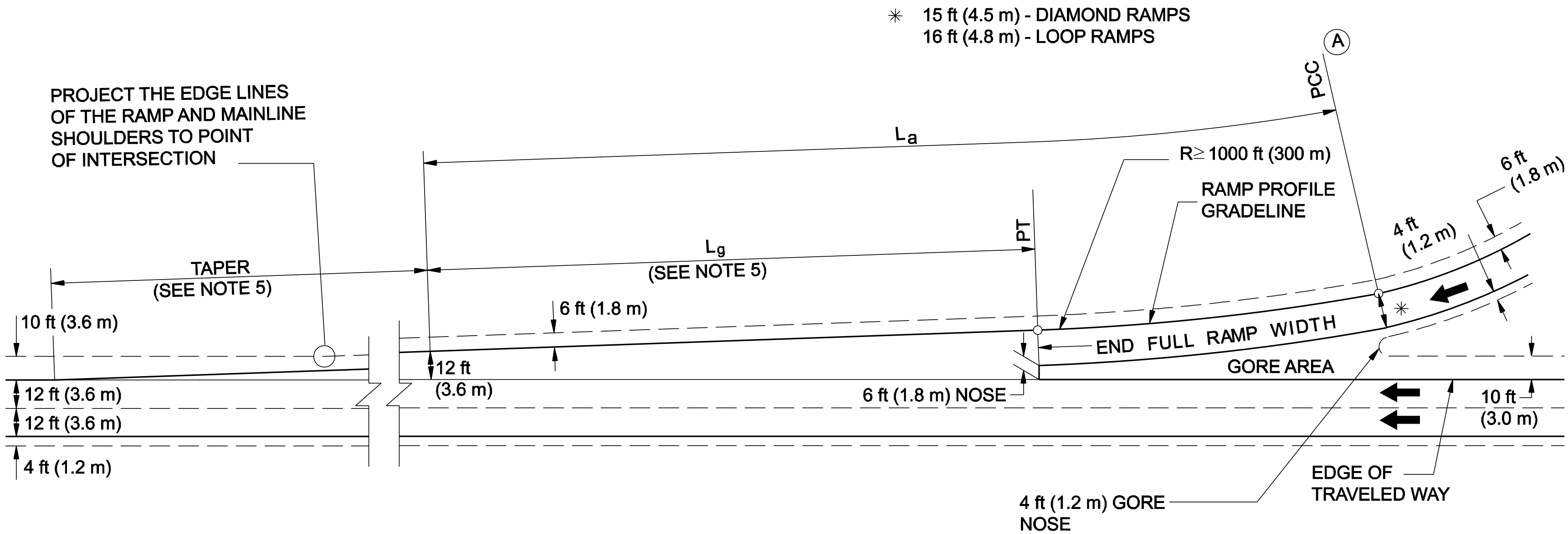
29.5.2.1 Types

There are two basic types of entrance freeway/ramp junctions — the taper design and the parallel design; see Figures 29.5G and 29.5H. For rural entrance ramps, the taper design should typically be used. For urban entrance ramps, the type of ramp will be determined on case-by-case basis considering the following:

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

2. Acceleration Length. Where the acceleration length needs to be lengthened for grades and or trucks, the parallel design provides longer distances more easily than a 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.

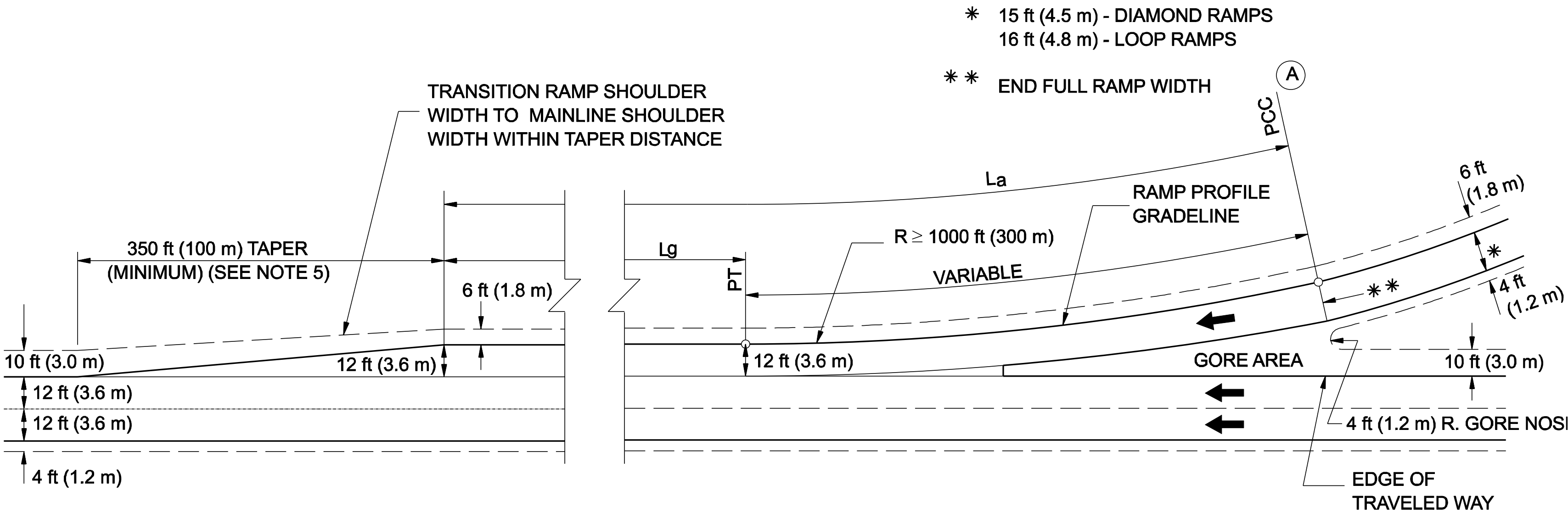


Notes:

1. *L_a is the required acceleration length; see Section 29.5.2.3.*
2. *Point (A) controls the safe speed on the ramp. L_a should not start on the curvature of the ramp unless the ramp radius is ≥1000 ft (300 m).*
3. *L_g is the required gap acceptance length. L_g should be a minimum of 300 ft to 500 ft (90 m to 150 m) from the 6 ft (1.8 m) nose width.*
4. *Use the greater distance of L_a or L_g for determining the ramp entrance length.*
5. *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 29.5G



Notes:

1. *L_a is the required acceleration length; see section 29.5.2.3.*
2. *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 ft (300 m).*
3. *L_g is the required gap acceptance length. L_g should be a minimum of 350 ft (100 m).*
4. *Use the greater distance of L_a or L_g for determining the ramp entrance length.*
5. *The taper rate should be 50:1 to 70:1 if L_g is 1200 ft (360 m) or greater.*

PARALLEL ENTRANCE RAMP

Figure 29.5H

4. Auxiliary Lane. Where there is a need for a continuous auxiliary lane, the parallel-lane entrance can be easily incorporated into the design of the continuous auxiliary lane.

Figures 29.5G and 29.5H provide the detailed design information for the Department's freeway/ramp entrances.

29.5.2.2 Taper Rates

The following taper rates apply to the entrance design:

1. Taper Design. This rate applies to the rate at which the ramp connects with the mainline. Typically, this will be at 60:1; see Figure 29.5G. However, the rate may be between 50:1 and 70:1.

2. 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 350 ft (100 m) as illustrated in Figure 29.5H.

29.5.2.3 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 level-of-service, 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 29.5I provides the minimum lengths of acceleration for passenger cars. The acceleration distance is measured from the PT of the last controlling curve to the beginning of the taper; see Figures 29.5G and 29.5H. Where upgrades exceed 3% over the acceleration distance, adjust the acceleration length according to the values presented in Figure 29.5J.

   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 ft (360 m), 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, consider providing the truck acceleration distances shown in Figure 29.5K. 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, the truck acceleration distances should be considered where there is substantial entering truck traffic and where:

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

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

   Where upgrades exceed 3%, truck acceleration distances may be corrected for grades. Figure 26.2E provides the performance criteria for trucks on accelerating 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. The specific application of the acceleration criteria to horizontal curves is as follows:

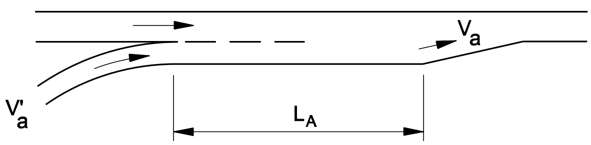
   a. The design speed of the last horizontal curve on the ramp proper will be determined by open-highway conditions. These are discussed in Chapter Nine of the Montana Road Design Manual. At a minimum, the curve on the ramp before the freeway/ramp junction should have a radius of at least 1000 ft (300 m).

   b. For relatively short entrance ramps, the acceleration distance may be determined 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.

29.5.2.4 Sight Distance

Drivers on the mainline approaching an entrance terminal should be provided sufficient distance to see the merging traffic so 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.

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	45	50
		And Initial Speed (mph) (V'_a)								
		0	14	18	22	26	30	36	40	44
30	23	180	140							
35	27	280	220	160						
40	31	360	300	270	210	120				
45	35	560	490	440	380	280	160			
50	39	720	660	610	550	450	350	130		
55	43	960	900	810	780	670	550	320	150	
60	47	1200	1140	1100	1020	910	800	550	420	180
65	50	1410	1350	1310	1220	1120	1000	770	600	370
70	53	1620	1560	1520	1420	1350	1230	1000	820	580
75	55	1790	1730	1630	1580	1510	1420	1160	1040	780



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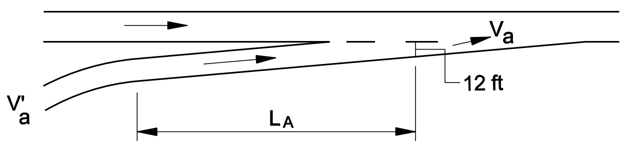


TAPER TYPE

Notes:

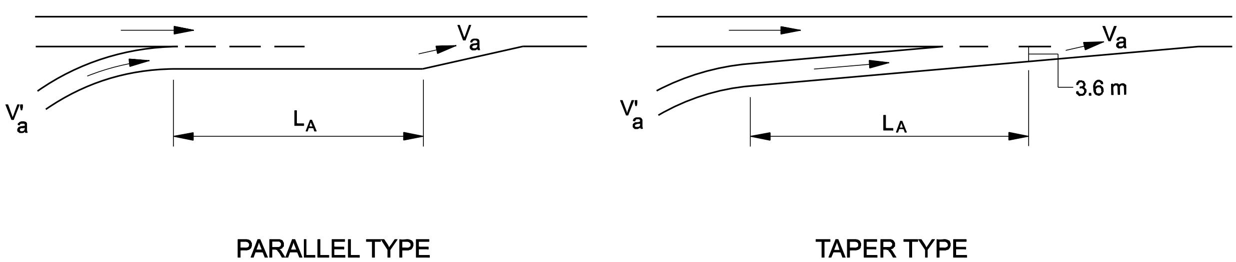
1. *The acceleration lengths are calculated from the distance needed for a passenger car to accelerate from the average running speed of the entrance curve to a speed of 5 mph below the average running speed on the mainline.*

2. *These values are for grades less than 3%. See Figure 29.5J for steeper upgrades.*

LENGTHS FOR ACCELERATION
(Passenger Cars)
(US Customary)

Figure 29.5I

Highway Design Speed (km/h) (V)	Speed Reached (km/h) (V_a)	L_A = Acceleration Length (m)							
		For Entrance Curve Design Speed (km/h)							
		Stop	20	30	40	50	60	70	80
		And Initial Speed (km/h) (V'_a)							
		0	20	28	35	42	51	63	70
50	37	60	50	30	-	-	-	-	-
60	45	95	80	65	45	-	-	-	-
70	53	150	130	110	90	65	-	-	-
80	60	200	180	165	145	115	65	-	-
90	67	260	245	225	205	175	125	35	-
100	74	345	325	305	285	255	205	110	40
110	81	430	410	390	370	340	290	200	125
120	88	545	530	515	490	460	410	325	245



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 a speed of 10 km/h below the average running speed on the mainline.*
2. *These values are for grades less than 3%. See Figure 29.5J for steeper upgrades.*

LENGTHS FOR ACCELERATION
(Passenger Cars)
(Metric)

Figure 29.5I

US Customary				
Design Speed of Highway (mph)	Design Speed on Ramp (mph)			
	20	30	40	50
	3% to 4% upgrade			
40	1.3	1.3	-	-
45	1.3	1.35	-	-
50	1.3	1.4	1.4	-
55	1.35	1.45	1.45	-
60	1.4	1.5	1.5	1.6
65	1.45	1.55	1.6	1.7
70	1.5	1.6	1.7	1.8
	5% to 6% upgrade			
40	1.5	1.5	-	-
45	1.5	1.6	-	-
50	1.5	1.7	1.9	-
55	1.6	1.8	2.05	-
60	1.7	1.9	2.2	2.5
65	1.85	2.05	2.4	2.75
70	2.0	2.2	2.6	3.0
Metric				
Design Speed of Highway (km/h)	Design Speed on Ramp (km/h)			
	40	50	60	70
	3% to 4% upgrade			
60	1.3	1.4	1.4	-
70	1.3	1.4	1.4	1.5
80	1.4	1.5	1.5	1.5
90	1.4	1.5	1.5	1.5
100	1.5	1.6	1.7	1.7
110	1.5	1.6	1.7	1.7
120	1.5	1.6	1.7	1.7
	5% to 6% upgrade			
60	1.5	1.5	-	-
70	1.5	1.6	1.7	-
80	1.5	1.7	1.9	1.8
90	1.6	1.8	2.0	2.1
100	1.7	1.9	2.2	2.4
110	2.0	2.2	2.6	2.8
120	2.3	2.5	3.0	3.2

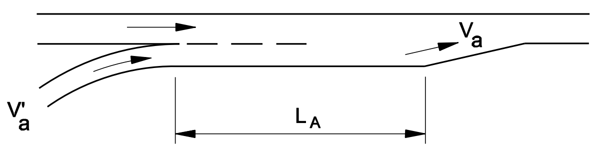
Notes:

1. *No adjustment is needed on grades less than 3%.*
2. *The "grade" in the table is the average grade measured over the distance for which the acceleration length applies. See Figures 29.5G and 29.5H.*

GRADE ADJUSTMENTS FOR ACCELERATION LENGTHS

Figure 29.5J

US Customary								
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	625	605	585	560	540	215
60	42	1365	1290	1270	1250	1225	1205	880
65	45	2055	1980	1960	1940	1915	1895	1570
70	48	2870	2795	2770	2750	2730	2710	2385
75	50	3500	3425	3405	3385	3360	3340	3015
Metric								
Highway Design Speed (km/h) (V)	Speed Reached (km/h) (V_a)	L_A = Acceleration Length (m)						
		For Entrance Curve Design Speed (km/h)						
		Stop	30	40	50	60	70	80
		For Average Running Speed (km/h) (V'_a)						
		0	30	40	47	55	63	70
90	61	235	205	195	185	95		
100	69	435	410	400	390	295	165	
110	75	680	655	645	635	540	410	205
120	83	1180	1150	1140	1130	1040	905	705



PARALLEL TYPE

Notes:

1. *The acceleration lengths are calculated from the distance needed for a 200 lb/hp (120 kg/kW) truck to accelerate from the average running speed of the entrance curve to reach a speed (V_a) that is 5 mph (15 km/h) below the average running speed on the mainline.*
2. *The taper entrance ramp is generally not applicable where trucks govern the design.*
3. *Below 55 mph (90 km/h), the minimum lengths for passenger cars in Figure 29.5I will apply.*

LENGTHS FOR ACCELERATION
(200 lb/hp (120 kg/kW) Truck)

Figure 29.5K

* * * * * * * * * *

Example 29-5.2

Given:	Highway Design Speed	-	70 mph
	Entrance Ramp Curve Design Speed	-	40 mph
	Average Grade	-	5% upgrade

Problem: Determine length of acceleration required.

Solution: Figure 29.5I yields an acceleration length of 1000 ft on a level grade. According to Figure 29.5J, this should be increased by a factor of 2.6 for a 5% upgrade.

Therefore: L = 1000 x 2.6
L = 2600 ft

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

* * * * * * * * *

29.5.2.5 Superelevation

The entrance ramp superelevation should be gradually transitioned to meet the normal cross slope of the mainline. The principles of superelevation for open-roadway conditions, as discussed in Chapter Nine of the Montana Road Design Manual, should be applied to the entrance design. Section 29.5.1.5 provides the superelevation criteria for exit freeway/ramp junctions, which are also applicable to entrance freeway/ramp junctions. This includes e_{max}, superelevation rate, transition lengths and the axis of rotation.

29.5.2.6 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 4% - 5% beyond the physical nose. Between the gore nose and physical nose, the maximum cross slope rollover is 8%. See Section 29.5.2.8 for gore area definitions.

29.5.2.7 Shoulder Transitions

At entrance terminals, the right shoulder must be transitioned from the narrower ramp shoulder to the wider freeway shoulder (i.e., 6 ft to 10 ft (1.8 m to 3.0 m)). Figures 29.5G and 29.5H illustrate the typical shoulder transition.

29.5.2.8 Gore Area

Section 29.5.1.8 provides the definitions for various nose types that are within the 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 ft (m) (i.e., the point where the two paint lines meet).

2. <u>Physical Nose</u>. The physical nose has a dimensional width of 14 ft (4.2 m), which is the width of the inside ramp and freeway shoulders.

3. <u>Gore Nose</u>. The gore nose is where the outside edges of the ramp and mainline shoulders are 8 ft (2.4 m) apart. The gore nose is designed with a 4 ft (1.2 m) radius.

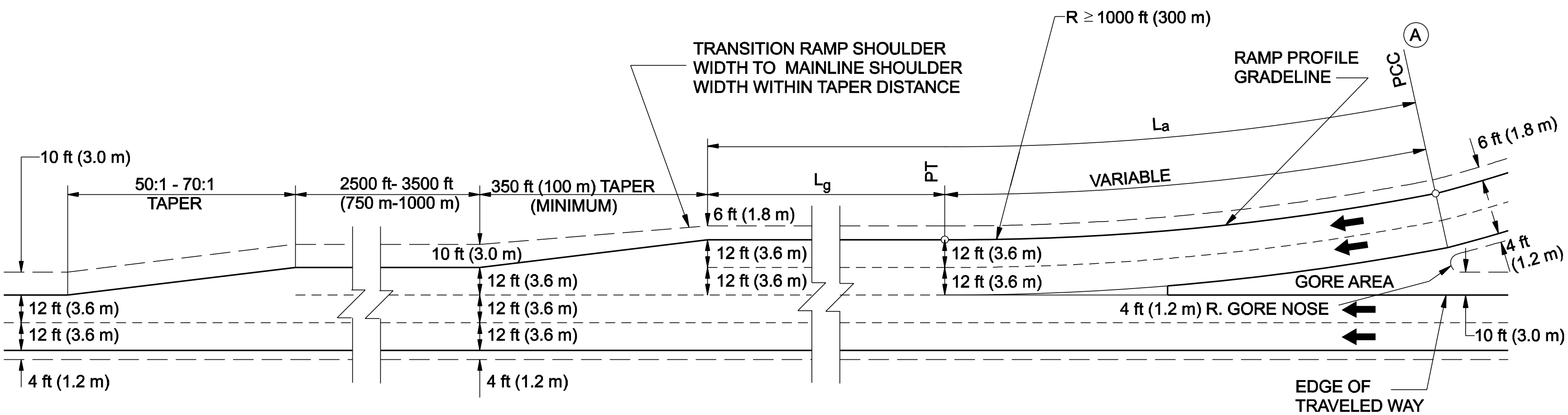
29.5.3 <u>Multilane Terminals</u>

Multilane terminals may be required when the capacity of the ramp is too great for a 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>. Lane balance at the freeway/ramp junction should be maintained; see Section 29.3.2.

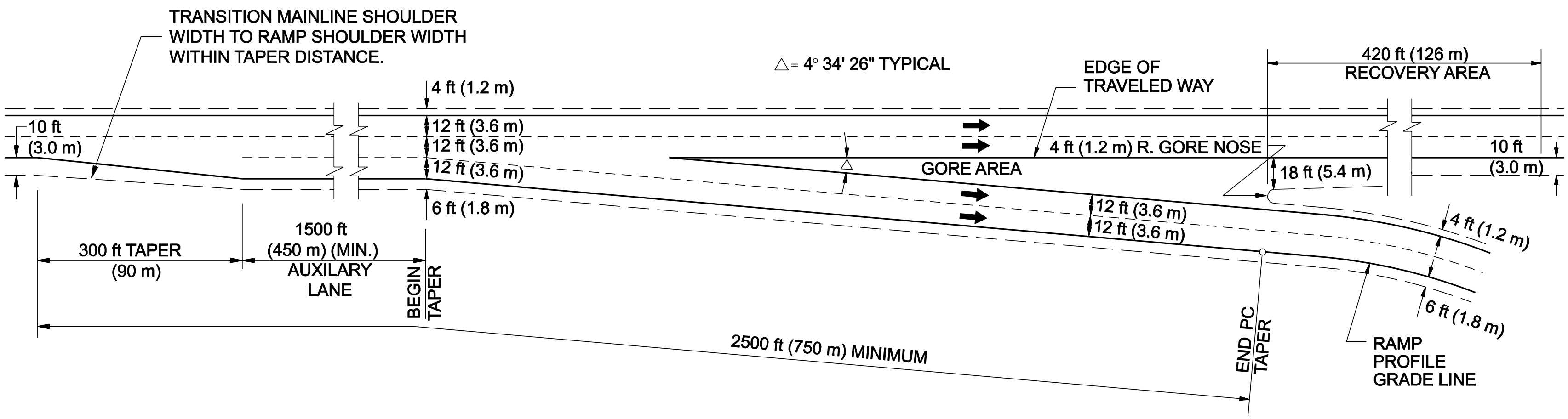
2. <u>Entrances</u>. For multilane entrance ramps, desirably a parallel-lane design should be used; however, a taper design is also acceptable. Figure 29.5L illustrates the parallel multilane entrance ramp.

3. <u>Exits</u>. For a 2-lane exit ramp, the additional lane should be added at least 1500 ft (450 m) prior to the terminal. The total length from the beginning of the first taper to the gore nose will range from 2500 ft (750 m) for turning volumes of 1500 vph or less up to 3500 ft (1000 m) for turning volumes of 3000 vph. Figure 29.5M illustrates a schematic of typical taper multilane exit ramps.



TWO-LANE ENTRANCE RAMP
(Parallel Design)

Figure 29.5L



TWO-LANE EXIT RAMPS

Figure 29.5M

4. Signing. Because of the complicated signing which may be required in advance of the exit, coordinate the geometric layout of multilane exits with the Signing and Pavement Marking Unit.

29.5.4 Major Fork/Branch Connections

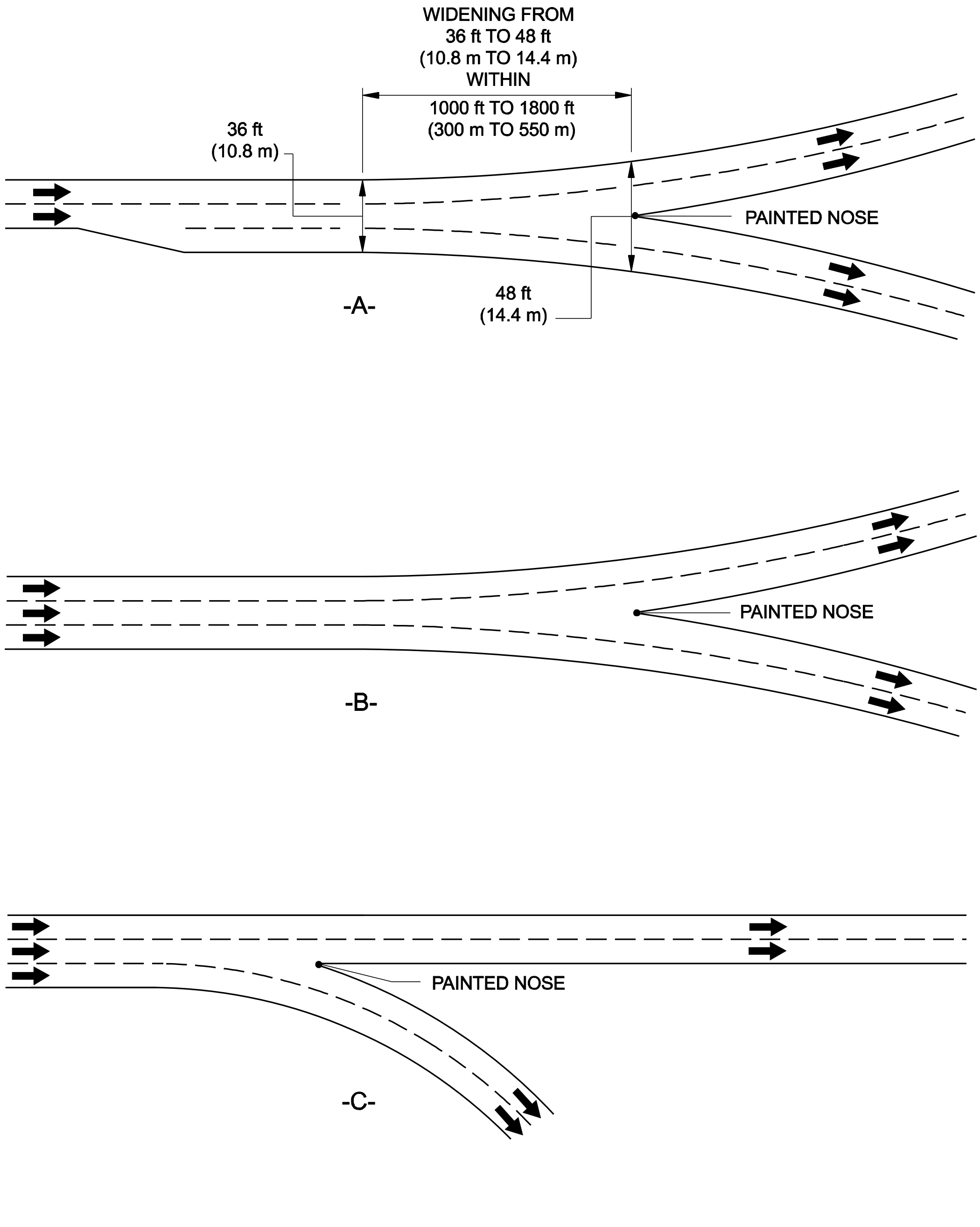
Figures 29.5N and 29.5O illustrate typical design details for a major fork or branch connection. The following presents a few geometric issues that the designer should consider when designing major divisions:

1. Lane Balance. The principle of lane balance should be maintained; see Section 29.3.2.

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 the driver in the center lane the option of going in either direction. See Schematics A and B in Figure 29.5N. 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 Schematic C in Figure 29.5N.

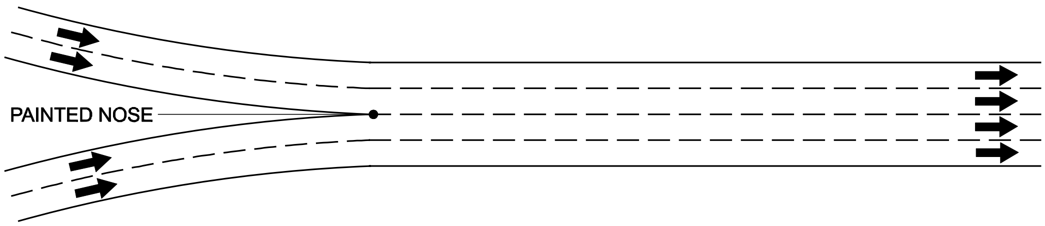
3. Nose Width. At the painted nose of a major fork, the lane should be at least 24 ft (7.2 m) wide but preferably not more than 28 ft (8.4 m). The widening from 12 ft to 24 ft (3.6 m to 7.2 m) should occur within a distance of 1000 ft to 1800 ft (300 m to 550 m). See Schematic A in Figure 29.5N.

4. Branch Connection. When merging, provide a full lane width for at least 1000 ft (300 m) beyond the painted nose. See Schematic B in Figure 29.5O.

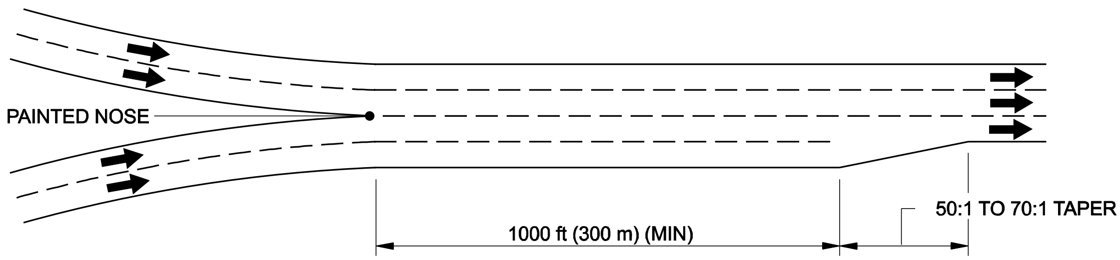


MAJOR FORKS

Figure 29.5N



-A-



-B-

BRANCH CONNECTIONS

Figure 29.5O

29.6 RAMP DESIGN

For design purposes, the ramp proper is assumed to begin or end at the gore nose.

29.6.1 Design Speed

Figure 29.6A provides the acceptable ranges of ramp design speed based on the design speed of the mainline. In addition, the designer should consider the following:

1. Freeway/Ramp Junctions. The design speeds in Figure 29.6A apply to the ramp proper and not to the freeway/ramp junction. Freeway/ramp junctions are designed using the freeway mainline design speed.
2. At-Grade Terminals. If a ramp will be terminated at an at-grade intersection with a stop or signal control, the design speeds in Figure 29.6A may not be applicable to the ramp portion near the intersection.
3. Variable Speeds. The ramp design speed may vary based on the two design speeds of the intersecting roadways. Use higher design speeds on the portion of the ramp near the higher speed facility and lower speeds near the lower speed facility. When using variable design speeds, the maximum speed differential between controlling design elements (e.g., horizontal curves, reverse curves)

US Customary						
Ramp Design Speed (mph)	Freeway Design Speed (mph)					
	50	55	60	65	70	75
Upper Range	45	48	50	55	60	65
Middle Range	35	40	45	45	50	55
Lower Range	25	28	30	30	35	40

Metric					
Ramp Design Speed (km/h)	Freeway Design Speed (km/h)				
	80	90	100	110	120
Upper Range	70	80	90	100	110
Middle Range	60	60	70	80	90
Lower Range	40	50	50	60	70

RAMP DESIGN SPEEDS

Figure 29.6A

should not be greater than 15 mph to 20 mph (20 km/h to 30 km/h). The designer must ensure that sufficient deceleration distance is available between design elements with varying design speeds (e.g., two horizontal curves).

4. Directional Ramps. Desirably, use a design speed in the upper range. These include both ramps at a diamond interchange and ramps at a directional interchange.

5. Semidirect Connections. Select a design speed in the middle and upper ranges.

6. 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 mph (40 km/h).

    b. Where the truck AADT is greater than 15%, consider using a minimum design speed of 30 mph (50 km/h) for the initial curve.

    c. For rural loop ramps, a 30 mph (50 km/h) design speed is preferred.

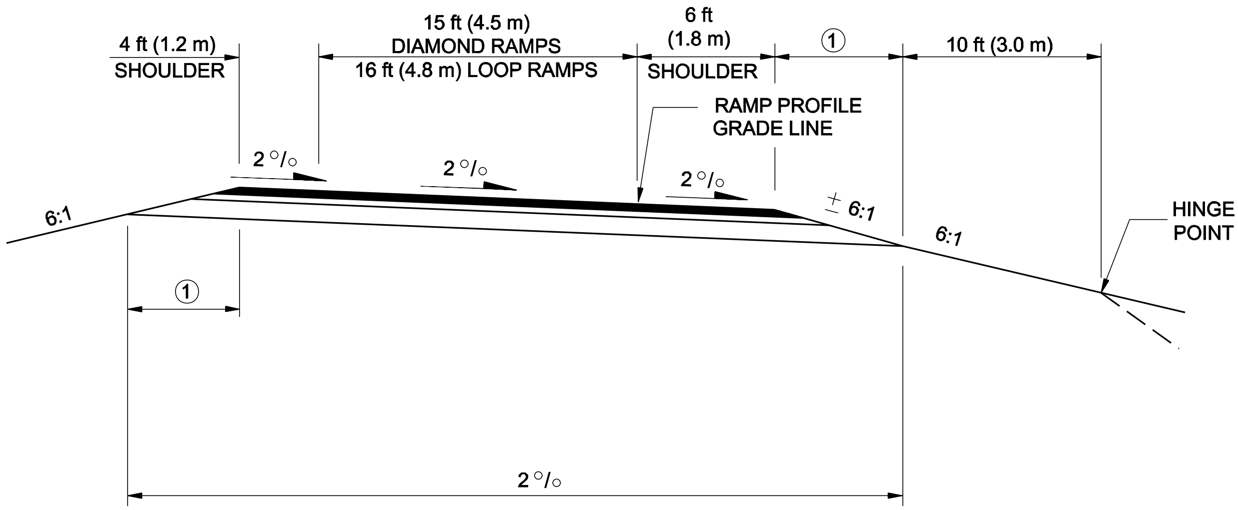
    d. Use a design speed of 35 mph (60 km/h) for cloverleaf interchange loop ramps between two freeways.

7. Outer Connection Ramps. The design speed for the outer connection ramp of a cloverleaf interchange should desirably be 45 mph to 50 mph (70 km/h to 80 km/h), but may be a minimum of 35 mph (60 km/h).

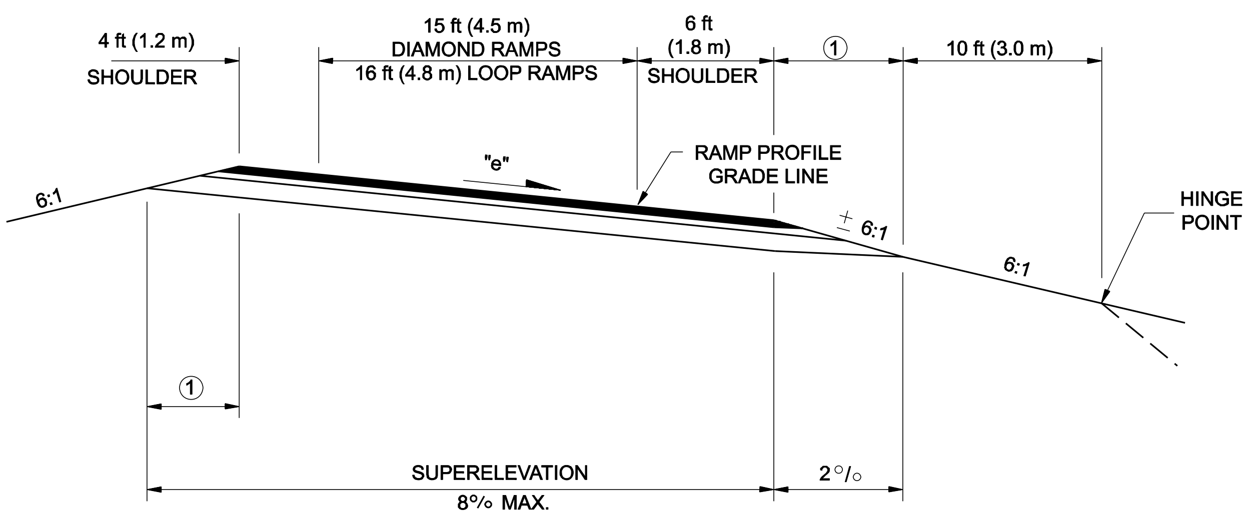
29.6.2 Cross Section

Figure 29.6B presents the typical cross sections for tangent and for superelevated ramps. The following will also apply to the ramp cross section:

1. Width. The minimum paved width of a one-way, 1-lane ramp will depend on the type of ramp. Diamond ramps will be 25 ft (7.5 m) wide and loop ramps 26 ft (7.8 m) wide. These widths include a 4 ft (1.2 m) left shoulder and a 6 ft (1.8 m) right shoulder. The traveled way portion for diamond ramps will be 15 ft (4.5 m) and, for loop ramps, 16 ft (4.8 m). This arrangement is illustrated in Figure 29.6B. For multilane ramp widths, see the ramp width criteria presented in AASHTO A Policy on Geometric Design of Highways and Streets (typically, use Case II, Condition B or Case III, Condition B).



TANGENT SECTION



SUPERELEVATED SECTION

① *Compute total width to nearest 0.1 ft (0.1 m). Compute intermediate surfacing widths to nearest 0.01 ft (0.01 m).*

TYPICAL RAMP CROSS SECTION

Figure 29.6B

2. Pavement Design. Ramp surfacing will correspond to that provided on the crossroad. The mainline structural section will be retained up to the gore nose. If the crossroad is unpaved, the ramp paving will end at the finished shoulder of the crossroad.

3. Cross Slope. For both tangent and superelevated sections, use a uniform cross slope across the entire ramp width. This includes both the left and right shoulders. For tangent sections, use a typical cross slope of 2%.

4. Curbs. In general, do not use curbs on ramps. However, where necessary, curbing may be used for drainage, to prevent erosion on steep embankment slopes or to separate adjacent on/off ramps provided that they are placed on the outside edge of the shoulder. Where the ramp design speed is less than or equal to 45 mph (70 km/h), either sloping on vertical curbing may be used. Where the ramp design speed is greater than 45 mph (70 km/h), only sloping curbing may be used.

5. Bridges and Underpasses. Carry the full paved approach width of the ramp over a bridge or beneath an underpass. The clear width under an underpass should also include the clear zone.

6. Side Slopes/Ditches. Side slopes and ditches should meet the same criteria as for the mainline. Chapter Twelve of the Montana Road Design Manual present the criteria for the design of these elements.

7. Clear Zones. The clear zone from the edge of the traveled way portion of the ramp will be determined from Chapter Fourteen of the Montana Road Design Manual. The design AADT will be the directional AADT on the ramp.

8. Barriers. Where a barrier is present on a horizontal curve, the designer should determine the barrier's impact on horizontal sight distance; see Section 25.5.

9. Right-of-Way. The right-of-way adjacent to the ramp should be limited access right-of-way.

29.6.3 Horizontal Alignment

29.6.3.1 Theoretical Basis

Establishing horizontal alignment criteria for any highway element requires a determination of the theoretical basis for the various alignment factors. These include the side-friction factor (f), the distribution method between side friction and superelevation, and the distribution of the superelevation transition length between the tangent and horizontal curve. For horizontal alignment on the ramp proper, the theoretical basis will be one of the following:

1. Open-Roadway Conditions. Chapter Nine of the Montana Road Design Manual discusses the theoretical basis for horizontal alignment assuming open-roadway conditions. In summary, this includes:

    a. relatively low side-friction factors (i.e., a relatively small level of driver discomfort);

    b. the use of AASHTO Method 5 to distribute side friction and superelevation;

    c. relatively flat longitudinal gradients for superelevation transition lengths; and

    d. typically distribute 70% of the superelevation transition length to the tangent and the remainder to the horizontal curve.

2. Turning-Roadway Conditions. Section 28.5.2.2 discusses the theoretical basis for horizontal alignment assuming turning-roadway conditions. In summary, this includes:

    a. higher side-friction factors than open-roadway conditions to reflect a higher level of driver acceptance of discomfort;

    b. a range of acceptable superelevation rates for combinations of curve radii and design speeds to reflect the need for flexibility to meet field conditions for turning roadway design; and

    c. the allowance of some flexibility in superelevation transition lengths and in the distribution between the tangent and curve.

For interchange ramps, the selection of which theoretical basis to use will be based on the portion of the ramp under design. The following sections discuss the horizontal alignment criteria for ramps.

29.6.3.2 General Controls

The following will apply to the horizontal alignment for ramp elements:

1. Superelevation Rates (Rural). For ramps in rural areas, the superelevation rate will be based on an e_{max} = 8% and open-roadway conditions. See Chapter

Twenty-five for specific superelevation rates based on ramp design speed and curve radius.

2. Superelevation Rates (Urban). For ramps in urban areas, the superelevation rate will be based on an e_{max} of 8%. Desirably, open-roadway conditions will be used. However, it will be acceptable to assume turning roadway conditions. See Chapter Twenty-five for specific criteria for open-roadway conditions. For turning-roadway conditions, see Section 28.5.2.2.

3. Superelevation Transitions. Desirably, the open-roadway conditions, as discussed in Chapter Twenty-five, will apply for transitioning to and from the needed superelevation on ramps. This includes the maximum relative longitudinal gradients.

4. Minimum Length of Design Superelevation. The designer should not superelevate curves on ramps so that the design superelevation rate is maintained on the curve for a very short distance. As a general rule, the minimum distance for design superelevation should be approximately 100 ft (30 m).

5. Axis of Rotation. This will typically be about the outside edge of the ramp traveled way.

6. Shoulder Superelevation. The criteria presented in Chapter Twenty-five for superelevating the high side and low side of shoulders on open roadways will apply to superelevated curves on ramps. The entire ramp width will have the same cross slope (i.e., it will be a plane).

7. Reverse Curves. To meet restrictive right-of-way requirements, ramps may be designed with reverse curves (e.g., for the outer connection of cloverleafs). Desirably, these reverse curves should be designed with a normal tangent section between the curves. For ramps, however, it is often necessary to provide a continuously rotating plane between the reverse curves. If a continuously rotating plane is used, the distance between the PT and the succeeding PC should desirably be 200 ft (60 m). It is acceptable for the PT and PC to be coincident. See Section 29.6.3.8 for more information on superelevation at reverse curves.

8. Sight Distance. Chapter Twenty-five presents the criteria for sight distance around horizontal curves based on the curve radii and design speed. These criteria also apply to curves on ramps.

29.6.3.3 Freeway/Ramp Junctions

Horizontal alignment at freeway/ramp junctions is based on open-roadway conditions. This is discussed in Section 29.5.

29.6.3.4 Ramp Proper (Directional Ramps)

Directional ramps refer to those ramps that are relatively direct in their alignment. These include ramps at diamond interchanges, the outer ramps at cloverleaf interchanges and ramps at directional and semi-directional interchanges. See the discussion in Section 29.6.3.2 to determine where open-roadway conditions or turning-roadway conditions apply to the horizontal alignment on directional ramps.

29.6.3.5 Ramp Proper (Loop Ramps)

Loop ramps are those ramps on the interior portions of cloverleaf and partial cloverleaf interchanges. Because of the normally restrictive conditions for loop ramps, the curve radii are typically less than 350 ft (100 m). Although it is desirable to use open-roadway conditions for horizontal alignment, typically, it is more practical to use turning-roadway conditions.

29.6.3.6 Ramp Terminus (Intersection Control)

Interchange ramps typically end at an at-grade intersection. The intersection may be stop control or signal control. If horizontal curves on the ramps are relatively close to the intersection, select a design speed for the curve that is appropriate for the expected operations at the curve. For these curves, the radius will determine whether open-roadway or turning-roadway conditions apply. For $R \geq 350$ ft (100 m), use open-roadway conditions. For $R < 350$ ft (100 m), open-roadway conditions are desirable; turning-roadway conditions are acceptable.

29.6.3.7 Ramp Terminus (Merge Control)

Interchange ramps may terminate with a merge into the intersecting road. The horizontal alignment at the ramp merge (or junction) will typically be based on open-roadway conditions.

29.6.3.8 Reverse Curves

Reverse curves are two closely spaced horizontal curves with deflections in opposite directions and a short, intervening tangent. For this situation, it may not be practical to achieve a normal crown section between the two curves. A plane section continuously rotating about its axis (i.e., the two inside edges of the ramp traveled way) can be used between the two curves, if they are sufficiently close together. The designer should adhere to the applicable superelevation development criteria (e.g., superelevation transition lengths) for each curve. The following will apply to reverse curves:

1. Normal Section. The designer should not attempt to achieve a normal tangent section between reverse curves unless the normal section can be maintained for a minimum distance of 100 ft (30 m), and the superelevation transition requirements can be met for both curves.

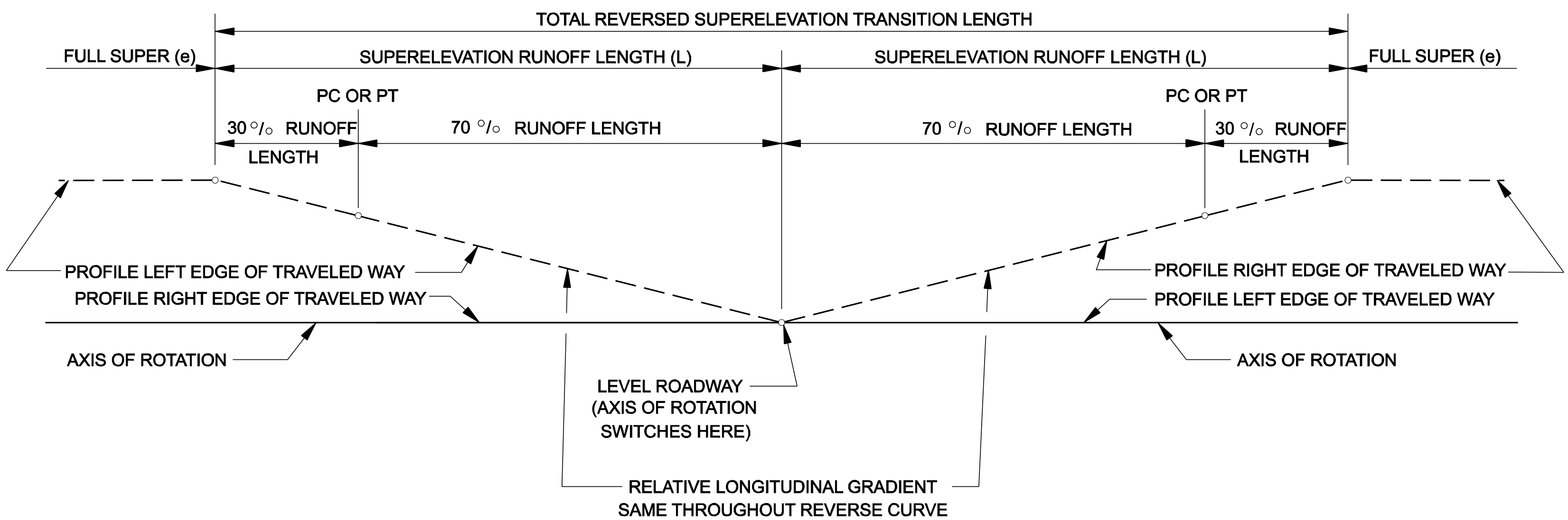
2. Continuously Rotating Plane. If a normal section is not provided, the pavement will be continuously rotated in a plane about its axis. In this case, the minimum distance between the ST and TS (or PT and PC) will be that needed to meet the superelevation transition requirements for the two curves. See Figure 29.6C for a schematic of a continuously rotating plane through a reverse curve. Note that, as illustrated in Figure 29.6C, the axis of rotation switches from one edge of traveled way to the other edge at the point where the roadway becomes level.

29.6.3.9 Bridges

From the perspective of the roadway user, a bridge is an integral part of the roadway system and, ideally, horizontal curves and their transitions will be located irrespective of their impact on bridges. However, practical factors in bridge design and bridge construction warrant consideration in the location of horizontal curves at bridges. The following presents, in order from the most desirable to the least desirable, the application of horizontal curves to bridges:

1. The most desirable treatment is to locate the bridge and its approach slabs on a tangent section and sloped at the typical cross slope (i.e., no portion of the curve or its superelevation development will be on the bridge or bridge approach slabs).

2. If a horizontal curve is located on a bridge, transitions should not be located on the bridge or its approach slabs. This includes both superelevation transitions and spiral transitions. This will result in a uniform cross slope (i.e., the design superelevation rate) and a constant rate of curvature throughout the length of the bridge and bridge approach slabs.



SUPERELEVATION OF REVERSE CURVES
(Continuously Rotating Plane)

Figure 29.6C

3. If the superelevation transition is located on the bridge or its approach slabs, the designer should place on the roadway approach that portion of the superelevation development that transitions the roadway cross section from its normal crown to a point where the roadway slopes uniformly. This will avoid the need to warp the crown on the bridge or the bridge approach slabs.

29.6.4 Vertical Alignment

29.6.4.1 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 29.6D, but for any one ramp the selected gradient is dependent upon a number of factors. These factors include the following:

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

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

4. Downgrades on ramps should follow the same guidelines as upgrades.

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.

Ramp Design Speed (km/h)	40	50	60	70	80
Ramp Design Speed (mph)	25	30	35	45	50
Maximum Grade Range (%)	5-7	5-7	4-6	3-5	3-5

RAMP GRADIENT GUIDELINES

Figure 29.6D

29.6.4.2 Vertical Curvature

Vertical curves on ramps should be designed similarly to those on the mainline. At a minimum, they should be designed to meet the stopping sight distance criteria. The ramp profile often assumes the shape of the letter S with a sag vertical curve at one end and with a crest vertical curve at the other. In addition, the vertical curvature of the ramp should be compatible with that of the mainline up to the physical nose. Where a crest or sag vertical curve extends onto the freeway/ramp junction, determine the length of curve using a design speed intermediate between those on the ramp and the highway. See Chapter Twenty-six for details on the design of vertical curves.

29.7 RAMP/CROSSROAD INTERSECTION

29.7.1 General Design Criteria

At diamond and partial cloverleaf interchanges, the ramp will terminate or begin with an at-grade intersection, either with a stop sign or a traffic signal. In general, the intersection should be designed as described in Chapter Twenty-eight. This will involve a consideration of capacity and physical geometric design elements (e.g., sight distance, angle of intersection, acceleration lanes, channelization, 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 Twelve of the Montana Road Design Manual.

2. Sight Distance. Section 28.9 discusses the criteria for intersection sight distance. These criteria also apply to the ramp/crossroad intersection. 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 intersection sight distance 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 lengthened 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/crossing road intersection can adversely affect the operation of the ramp/freeway junction. In a worst-case situation, the safety and operation of the mainline itself may be impaired by a backup onto the freeway. 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 traffic signalization where the ramp traffic will have priority. The analysis must also consider the operational impacts of the traffic characteristics in either direction on the intersecting road.

4. Turn Lanes. Left- and/or right-turn lanes often will be required on the crossroad and in some cases on the ramp itself. Chapter Twenty-eight provides information on the design of turn lanes at intersections which are also applicable to ramps.

5. Signalization. Where queuing at one intersection is long enough to affect operations at another, the two intersections may require a larger separation, interconnected signals, or a four-phase overlap signal design.

6. Design Vehicle. Design all radius returns and left-turn control radii for ramp/crossroad intersections using a WB-67 (WB-20) design vehicle; see Section 28.2.2.

7. Typical Designs. Figures 29.7A and 29.7B illustrate typical ramp/crossroad intersections for a diamond interchange. Figure 29.7A illustrates a 2-lane crossroad and Figure 29.7B a 4-lane divided crossroad.

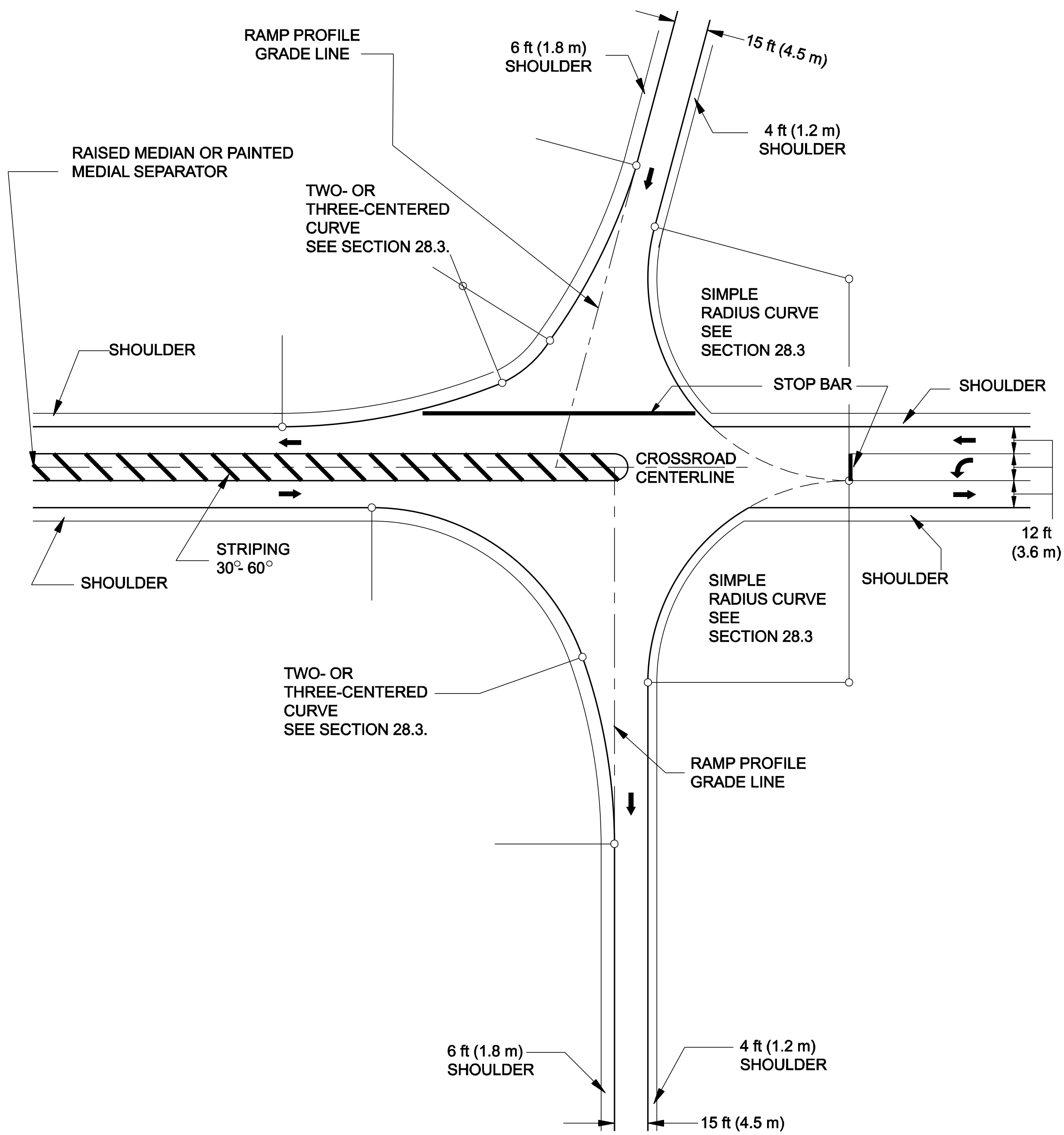
8. Wrong-Way Movements. Wrong-way movements may originate at the ramp/ crossroad intersection onto an exit ramp. To minimize the probability of these types of movements, design the intersection to discourage this movement and sign the existing ramp according to the criteria in the MUTCD and Chapter Eighteen.

9. Cattle Guards. Where cattle guards are required on ramps, place them approximately 150 ft (50 m) from the crossroad.

10. Crossroad Surfacing. The following will apply:

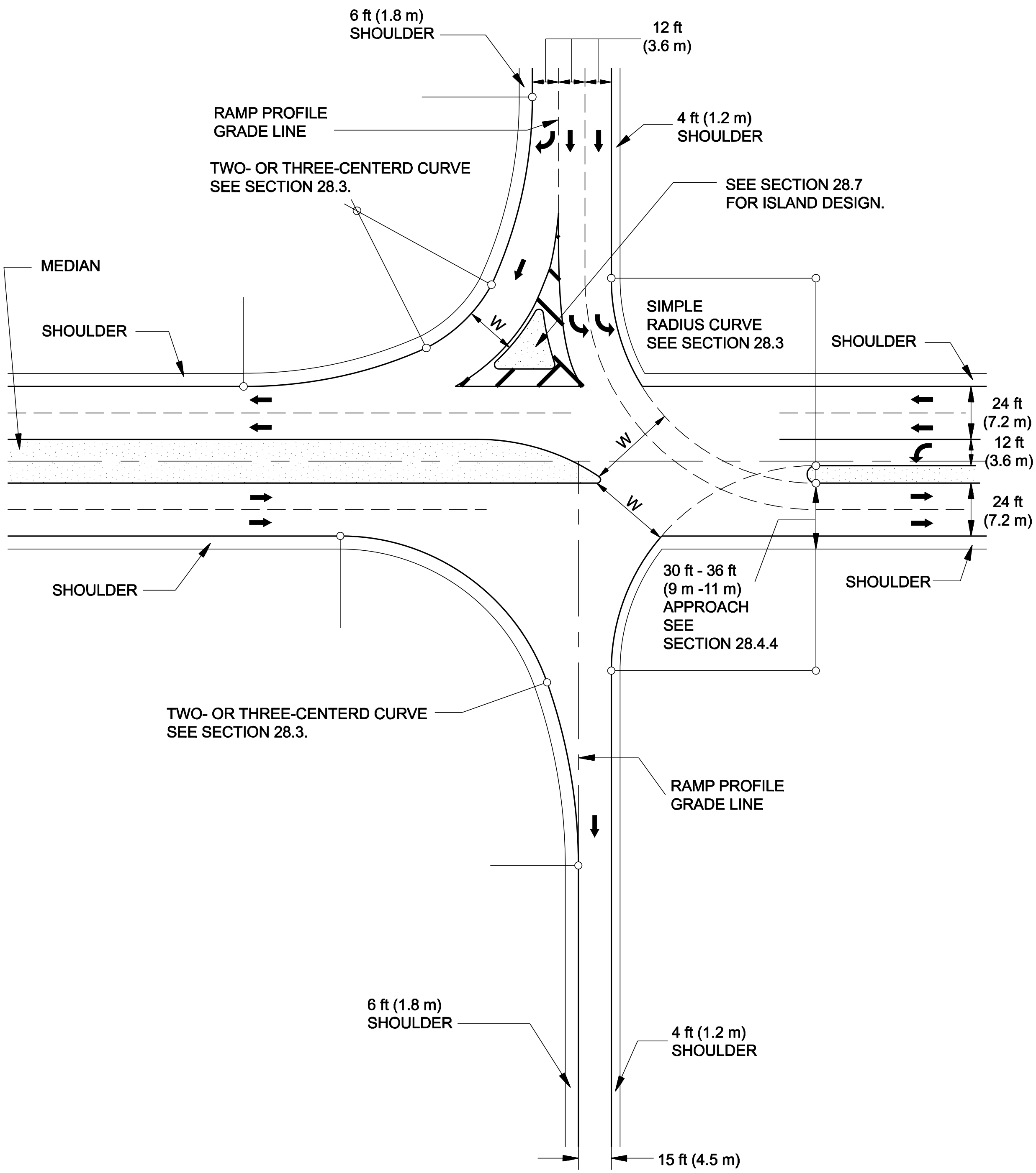
    a. Crossroad Over Freeway. Provide a minimum of 2 in of (60 mm) plant mix surfacing between and including the ramp terminals. From the ramp terminals to the ends of construction, the surfacing will correspond to the anticipated traffic in the design year.

    b. Crossroad Under Freeway. The surfacing will conform to the design year requirements for the crossroad. However, where the crossroad design year AADT is 100 or greater, provide at least 2 in (60 mm) of plant mix surfacing between and including the ramp terminals.



RAMP/CROSSROAD INTERSECTIONS — DIAMOND INTERCHANGE (2-Lane Crossroad)

Figure 29.7A



Note: For width of "W," see Section 28.5.2.3.

RAMP/CROSSROAD INTERSECTIONS — DIAMOND INTERCHANGE
(4-Lane Crossroads — Signalized Intersections)

Figure 29.7B

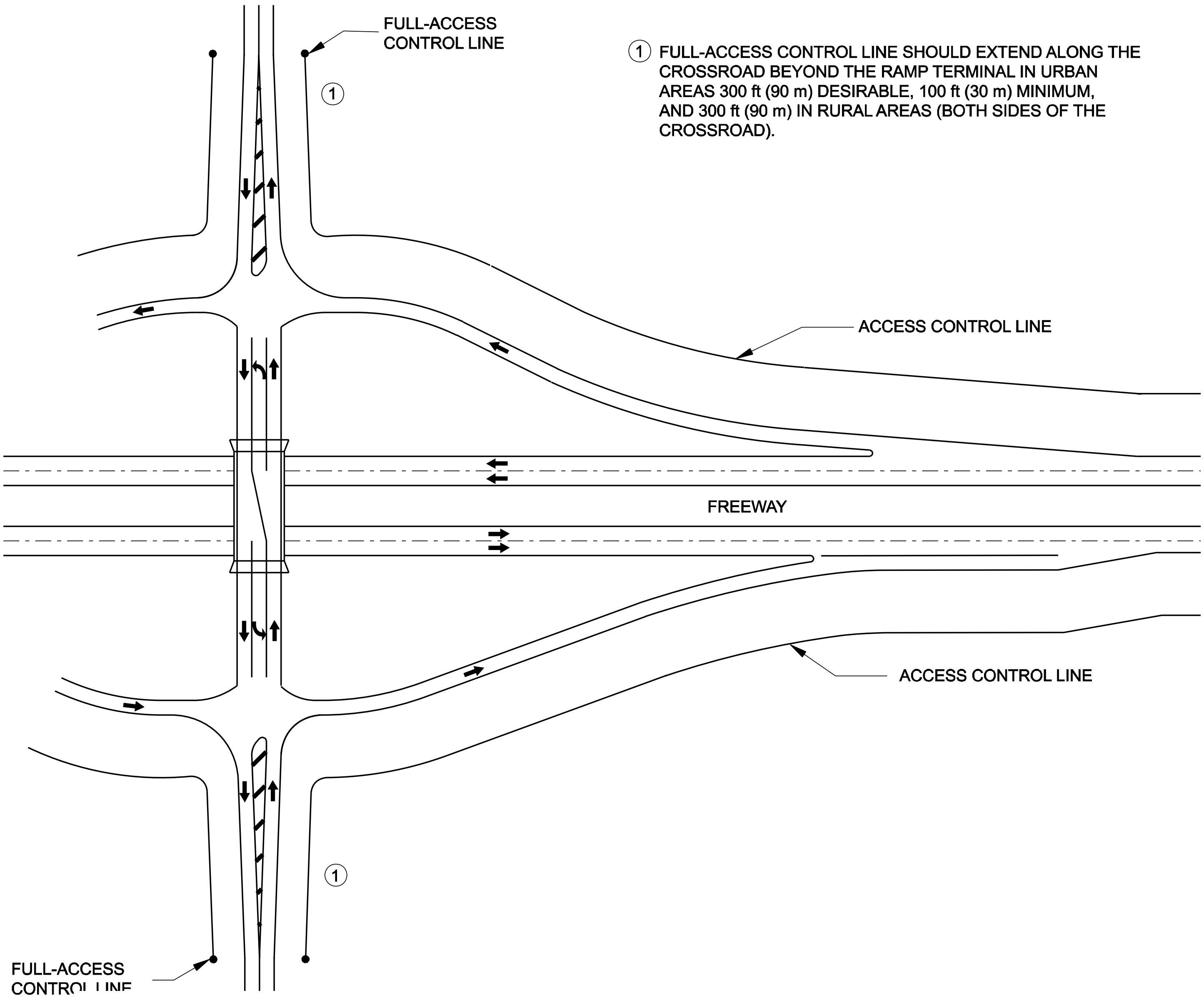
29.7.2 Access Control

Proper access control must be provided along the crossing road in the vicinity of the ramp/crossing road intersection or along a frontage road where present. This will ensure that the intersection has approximately the same degree of freedom and absence of conflict as the freeway itself. The access control criteria should be consistent with these goals.

Figures 29.7C and 29.7D illustrate the access control for diamond and partial cloverleaf interchanges. These figures provide MDT policy for the location of the full-access control lines along the ramp, at ramp/crossing road intersections, across from the ramp terminal and along frontage roads.

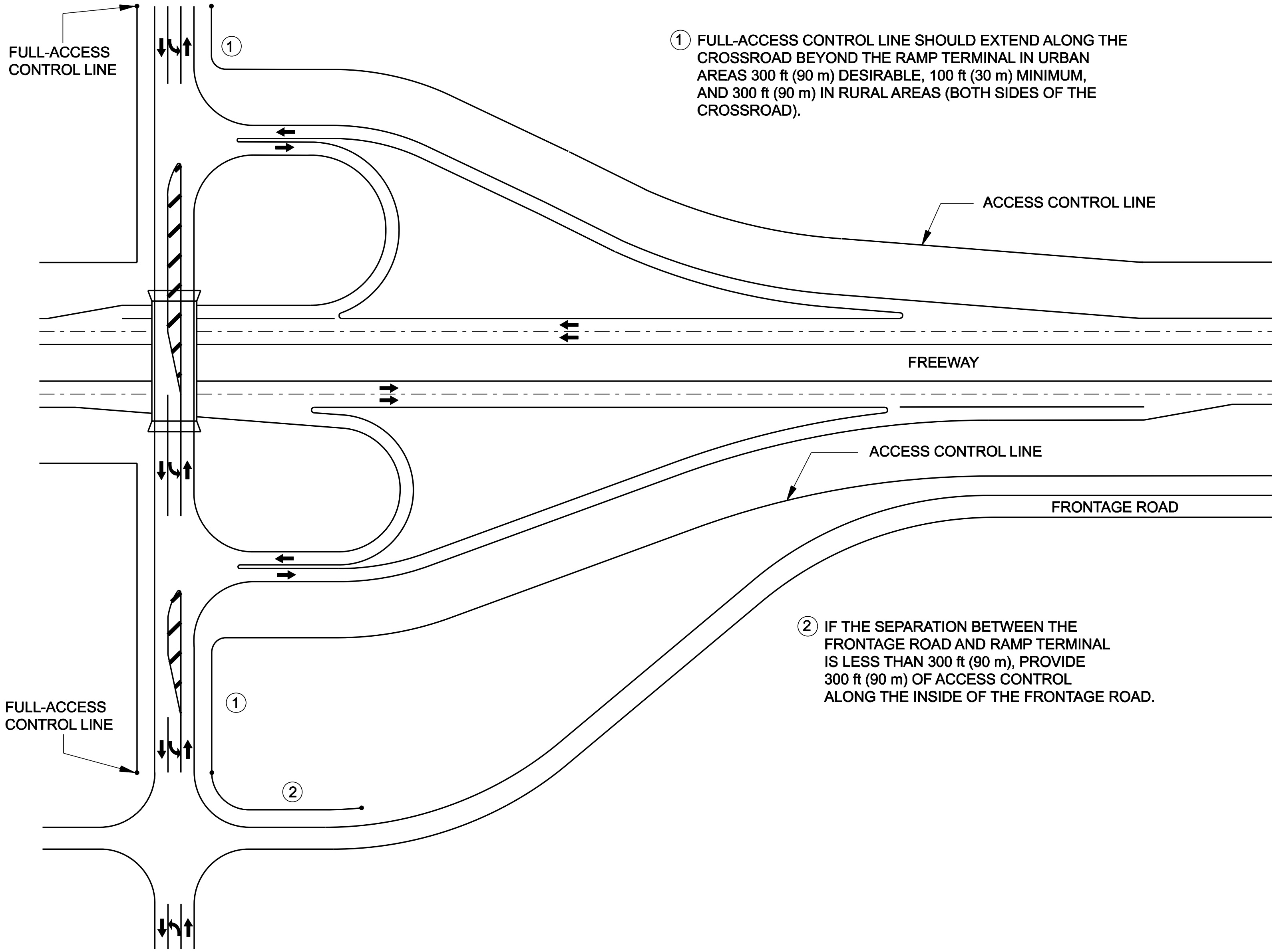
As indicated in the figures, the full-access control lines should extend 300 ft (90 m) in rural areas along the crossing road beyond the ramp or frontage road taper extremity on both sides of the road. In urban areas, desirably the full-access control line should extend 300 ft (90 m); the minimum distance is 100 ft (30 m). However, in areas where the potential for development exists that may present traffic problems, it may be appropriate to consider longer lengths of access control. In addition, many areas have changed over the years from rural to urban. As indicated, the Department has adopted different criteria for the access control at urban and rural interchanges. However, a change in area character alone is not a sufficient justification to alter the location of the full-access control line when an existing interchange will be rehabilitated or when the Department receives requests for additional access points from outside interests.

The figures note that, on the crossing road, the full-access control line should extend the indicated distance beyond “the ramp terminal.” For an exit ramp, this is defined as the tangent point (PT) of a radius return on the crossing road or the end of a taper for an entrance onto the crossing road (e.g., for an acceleration lane); i.e., the ramp terminal ends where the typical section of the crossing road resumes. A similar definition applies to ramp terminals for entrance ramps.



TYPICAL ACCESS CONTROL FOR A DIAMOND INTERCHANGE

Figure 29.7C



TYPICAL ACCESS CONTROL FOR A PARTIAL CLOVERLEAF INTERCHANGE

Figure 29.7D