

CHAPTER 48

Interchanges

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CHAPTER 48

INTERCHANGES

48-1.0 GENERAL

An interchange is a system of interconnecting roadways in conjunction with one or more grade separations that provides for the movement of traffic between two or more roadways on different levels.

48-1.01 INDOT Procedures

The Traffic Engineering Division Corridor Development Office is generally responsible for determining the need for, location of and type of interchanges. This assessment is based on a consideration of several factors, which are discussed in Sections 48-1.0 and 48-2.0. The designer is responsible for determining the layout and design of the interchange as discussed in Sections 48-3.0 through 48-6.0.

48-1.02 Guidelines

Although an interchange is a high-level compromise for intersection problems, its high cost and environmental impact require that an interchange be used only after careful consideration of its benefits. Because of the great variance in specific site conditions, INDOT has not adopted specific interchange warrants. Consider the following when determining the need for an interchange or grade separation:

1. Design Designation. Once it has been decided to provide a fully access-controlled facility, each intersecting highway must be terminated, rerouted, provided a grade separation or provided an interchange. The importance of the continuity of the crossing road and the feasibility of an alternative route will determine the need for a grade separation or interchange. An interchange should be provided on the basis of the anticipated demand for access to the minor road.

On facilities with partial control of access, intersections with public roads will be accommodated by an interchange or with an at-grade intersection; grade separations alone are not normally provided. Typically, an interchange will be selected for the higher-volume intersecting roads. Therefore, on a facility with partial control of access, the decision to provide an interchange will be, in general, based on the criteria in Section 48-1.04.

2. Congestion. An interchange may be considered where the level of service (LOS) at an at-grade intersection is unacceptable, and the intersection cannot be redesigned at-grade to operate at an acceptable LOS. Although LOS criteria is the most tangible of any interchange guideline, INDOT has not adopted any specific levels which, when exceeded, would demand an interchange. Even on facilities with partial control of access, the elimination of signalization contributes greatly to the improvement of flow.
3. Safety. The accident reduction benefits of an interchange should be considered at an existing at-grade intersection which has a high accident rate. The elimination of railroad-highway crossings should be considered in this factor. Section 48-3.08 provides additional information on various safety considerations relative to interchange selection.
4. Site Topography. At some sites the topography may be more adaptable to an interchange than an at-grade intersection.
5. Road-User Benefits. Interchanges significantly reduce the travel time when compared to at-grade intersections but may increase the travel distances. If an analysis reveals that road-user benefits over the service life of the interchange will exceed costs, then an interchange may be considered. For more information on road-user benefit analysis, see Chapter 50.
6. Traffic Volume. Interchanges should be considered at crossroads with heavy traffic volumes because elimination of conflicts greatly improves the movement of traffic.
7. Other Factors. Other factors, which need to be considered, include construction costs, right-of-way impacts and environmental concerns.

48-1.03 New or Revised Access to the Interstate System

48-1.03(01) Applicability

Each entrance or exit point to an Interstate freeway route is considered an access point. For example, a conventional diamond interchange has four access points, two on-ramps and two off-ramps. Locked-gate access is defined as an access point, and is described in Section 48-1.03(02), Item 9.

Revised access to an Interstate route is a change in the existing essential form, even though the sheer number of access points does not change. For example, adding a loop on-ramp in concert with a collector-distributor (C-D) roadway linked with a downstream diagonal on ramp to an otherwise conventional diamond interchange, or changing a cloverleaf interchange into a fully-

directional interchange is considered revised access. Lengthening or adding auxiliary lanes at at-grade ramp terminals with crossroads or ramp-proper lanes is not considering revised access, nor is converting a single-lane off- or on-ramp to dual-lanes. This is clarified in Sections 48-1.03(02) and 48-1.03(03).

The design of new or revised access must comply with AASHTO's *A Policy on Geometric Design of Highways and Streets* (AASHTO *GDHS*), AASHTO's *A Policy on Design Standards – Interstate System* (*Interstate Standards*), and this manual.

Work determined to consist of new or revised access to the existing Interstate System will require development by INDOT of a formal request to FHWA for New or Revised Access to the Interstate System. The Interstate Access Request, previously known as an Interchange Justification Report, requires the development of an associated Interstate Access Document (IAD). The IAD is a stand-alone document, which must accompany the request from INDOT to FHWA for approval of new or revised access. The IAD must document and demonstrate that reasonable care has been taken in addressing the criteria described in the FHWA Policy on Access to the Interstate System (FHWA Policy) as described in the *State of Indiana Interstate Access Request Procedures* and Section 48-1.03(06). The IAD must confirm that future traffic operations along the affected Interstate corridor will not be adversely affected by the proposed action. Revisions to the FHWA Interstate Access Policy occur periodically to ensure the focus remains on safety, operational, and engineering issues. The entire Interstate System in the state is under jurisdiction of INDOT. Only INDOT, and not a local public agency or private concern, may develop an Interstate Access Request and submit it to FHWA for approval. New or revised access to the Interstate System must be in accordance with the *State of Indiana Interstate Access Request Procedures*. The procedures are available from INDOT's Designers webpage at <http://www.in.gov/indot/2731.htm>.

The requirement for an Interstate Access Request and such FHWA approval applies only for non-tolled Interstate routes and Interstate toll roads where federal-aid funds have been expended or where the tolled sections have been added to the Interstate System under the requirements of 23 USC 139(a). Access to a non-Interstate System freeway or to a new Interstate System highway does not require an Interstate Access Request. INDOT has the authority to approve new or revised access to all other types of routes where federal-aid funds were used to acquire the access control. For this situation, INDOT must obtain the value of the access from the appropriate property owner(s) and either credit the federal share under existing disposal requirements, or determine that the net proceeds can be handled in accordance with 23 USC 156. INDOT may request FHWA advice or assistance on the acceptability of these types of new or revised access if desired.

48-1.03(02) Actions Requiring an Interstate Access Request

Actions that require INDOT to develop and FHWA to approve an Interstate Access Request include:

1. Establishing a new Interstate-to-Interstate or Interstate-to-freeway (system) interchange;
2. Major modification of a system interchange; e.g., adding new ramp(s), removing ramp(s) from service, significantly relocating tie-in points (terminals) on the freeway, or, where all movements are not currently accommodated, adding ramps to provide for all movements;
3. Upgrading an Interstate-to-non-freeway (service) interchange to an Interstate-to-freeway or Interstate-to-Interstate system interchange;
4. Establishing a new or revised partial interchange on the Interstate of any form;
5. Establishing a new Interstate-to-non-freeway (service) interchange;
6. Modifying an existing Interstate-to-non-freeway (service) interchange, e.g., adding a new ramp, removing a ramp from service, significantly relocating tie-in points (terminals) on mainline freeway or crossroad, or adding or significantly altering collector-distributor (C-D) road elements;
7. Removing select access points or ramps or an entire interchange from service;
8. Changing the essential type of interchange, e.g., replacing conventional diamond with partial cloverleaf;
9. Changing the essential form of a ramp, e.g., directional, semi-directional, loop, or diagonal;
10. Changing intersection control at ramp terminals where the change may affect mainline Interstate flow, even if a new access point to the Interstate is not created. For example, the conversion of a conventional diamond interchange to a diverging diamond interchange or single point diamond interchange is a change that may affect mainline Interstate flow.

11. New or revised locked-gate access or access via locked gates for privately or publicly employed personnel. Locked-gate access is limited to use by utility or INDOT personnel and not the general public; or
12. Establishing new or revised access not explicitly listed above, e.g., those rising to a level beyond incidental work. These instances should be coordinated with the Traffic Engineering Division Corridor Development Office.

48-1.03(03) Actions Not Requiring an Interstate Access Request

The following action do not require INDOT to develop an Interstate Access Request; however, traffic analysis to support the action is essential and should be included in the project file.

1. Changing a single-lane freeway exit or entrance to a two-lane freeway exit or entrance;
2. Widening a single-lane on- or off-ramp (ramp proper) to two or more lanes;
3. Widening (adding auxiliary lanes to) an on- or off-ramp at its intersection with a crossroad (at-grade terminal) to provide two or more intersection approach lanes;
4. Implementing traffic signal control at the ramp terminals;
5. Realigning ramp (minor changes to horizontal or vertical alignment);
6. Converting a tapered on-ramp design (single-lane or multi-lane) or a tapered single-lane off-ramp design to a parallel design.
7. Converting a parallel design off-ramp (single-lane or multi-lane) to a tapered design multi-lane off-ramp with option lane;
8. Increasing the length of an on-ramp acceleration lane or an off-ramp deceleration lane;
9. Adding one or more continuous auxiliary lanes between two adjacent interchange ramps. An operational analysis is required for this action. The NEPA process must be complied with for potential significant environmental impacts from the added lanes (noise, air quality, additional right-of-way, etc.); or
10. Other minor actions not explicitly listed above.

Regardless of the need for an Interstate Access Request, a traffic operational analysis should be conducted. INDOT will informally consult with the appropriate FHWA Transportation Engineer even if such project is not subject to FHWA oversight.

48-1.03(04) Programmatic Agreement for Interstate Access Requests

INDOT and FHWA entered into a Programmatic Agreement for Interstate Access Requests in October 2016. This agreement allows INDOT to conduct the necessary review and assessment of the justification and documentation substantiating certain proposed changes in Interstate System access. The agreement also allows INDOT to make a determination of engineering and operational acceptability (EOA) for proposed changes and request expedited FHWA approval.

INDOT's determination of EOA is limited to:

1. New and major modifications to existing freeway-to-crossroad (service) interchanges and
2. Completion of basic movements at existing partial interchanges.

The Programmatic Agreement does not include:

1. New or modified freeway-to-freeway (system) interchanges;
2. New partial interchanges;
3. Closure of individual access points that result in partial interchanges or closure of entire interchanges; and
4. Locked gate access.

The Programmatic Agreement and the related *State of Indiana Interstate Access Request Procedures* document are available from INDOT's Designers webpage at <http://www.in.gov/indot/2731.htm>.

48-1.03(05) Coordination with National Environmental Policy Act Requirements

When a federal agency is required to make an approval action, regardless of the funding source, the NEPA process must be followed. Since FHWA approves INDOT's Interstate Access Requests, the NEPA process must be followed when developing new or revised Interstate access. The NEPA process should proceed concurrently with development and analysis of (existing) Interstate access alternatives. The intention is to eliminate early alternatives that would not be acceptable from a transportation and safety operations standpoint. The final decision on a preferred and selected alternative is made as part of the NEPA process. FHWA final Interstate Access Request approval can only be obtained after completion of the NEPA process.

48-1.03(06) General Steps in Revising or Adding Access to the Interstate System

There are five sequential steps in the process for INDOT to secure authorization from FHWA to change Interstate System access. These proposed actions usually require an Environmental Impact Statement (EIS) or an Environmental Assessment (EA) to complete the NEPA process. The first two steps effectively take place as a forerunner to the formal Interstate Access Request process. The steps are outlined in the *State of Indiana Interstate Access Request Procedures* document and are summarized below.

1. Framework for Project Scope. Establish the framework for scope of study relative to alternatives' analysis, and record that in a concise *Framework Document*. At the start of access request process, the INDOT project team will meet with FHWA to identify any special process and operational requirements. The Traffic Engineering Division Corridor Development Office oversees development of all Interstate Access Request activities. The FHWA Project Delivery Team Leader will serve as INDOT's point of contact for this process of developing and screening alternatives. The FHWA Project Delivery Team's assigned Transportation Engineer will represent FHWA in providing opinion and review of alternatives from an engineering and transportation operations standpoint.
2. Alternatives Analysis and Selection. Carry out alternatives' analysis, and document those activities and findings in a report - the *Alternative Evaluation Report*. Its findings will indicate if the Interstate Access Request and associated Interstate Access Document (IAD) are required. If an Interstate Access Request is not required for an interchange modification project, an Alternative Evaluation Report will still be required to identify the site, background information, deficiencies, alternatives and proposals. The report will evaluate traffic operations and safety performance of each alternative regarding the interchange itself and the mainline interstate.
3. Interstate Access Request Determination. Determine whether an Interstate Access Request to FHWA and its associated IAD are required, and if so, prescribe the nature or scale of that IAD.
4. Draft Interstate Access Document Submittal. Produce the IAD, and transmit to FHWA from INDOT the request for engineering and operational acceptability along with that supporting IAD. The draft document will focus on the points of the FHWA Policy. FHWA's Concept Approval is given with the understanding that the proposal will be that which is reflected in the final NEPA document, either CE, Finding of No Significant Impact (FONSI), or Record of Decision (ROD). This is the first of two approval phases.
5. Request for Final Interstate Access Approval. Transmit to FHWA from INDOT the request for full and final approval, following NEPA approval (CE, FONSI, or ROD). FHWA will

respond in writing within four weeks indicating approval or denial of INDOT's formal request for new or revised access. This is the second of the dual approval phases.

48-1.03(07) Content of the Interstate Access Document

The Interstate Access Document (IAD) that accompanies the Interstate Access Request must address the policy requirements (criteria) outlined in the FHWA Policy. The Programmatic Agreement includes the criteria while the *State of Indiana Interstate Access Request Procedures* document outlines the analysis and documentation requirements for requesting changes to Interstate System access. The IAD serves as the record of that analysis in the form of answers to the FHWA Policy criteria.

The criteria will be the focus of attention in the IAD and must be directly addressed. Other background information may be presented to supplement that core element. A clear description of the proposed new or revised access should be presented, generally in narrative form directing the reader to sketch-plan drawings. All relevant notes, summary printouts, and/or electronic input/output files of traffic operation and safety analysis should be appended to the IAD document, be they from HCM / HCS, microsimulation, HSM / IHSDM or other method of analysis.

Background information should be included that may help explain or support the proposal, including a description of the influence of the area's regional transportation network, and any known areas of concern, e.g., environmental, safety, related projects, and long-range transportation plans. A crash analysis summary must be included. The analysis must include a summary of crash data for the previous three-year period. There must be a discussion of the anticipated safety impact the access change will have on the Interstate-route mainline and interchange ramps. The analysis must demonstrate that the access change will not compromise safety. The recommended alternative should include plans with, at a minimum, a table of basic geometric design criteria, horizontal and vertical alignment, curve data, typical sections, signing, and pavement markings. Any necessary design exceptions should desirably be identified. In addition, the total estimated cost of the project should be provided. A complex urban project may require a conceptual-stage signing plan if determined to be necessary by FHWA and INDOT.

48-1.03(08) FHWA Approval

Approval is required from the FHWA Washington, D.C., Headquarters office (HQ) for the major types of new or revised access requests listed below. The Final IAD must be sent by INDOT to the FHWA Indiana Division office for those actions of a significant nature requiring coordination with HQ. Advance coordination with HQ will be necessary for certain complex or controversial

projects. INDOT should coordinate directly with the Division office, specifically, the appropriate Transportation Engineer for all projects.

1. FHWA Approval by HQ. HQ approval is required for the types of Interstate System new or revised access as follows:
 - a. establishing a new freeway-to-freeway (system) interchange;
 - b. major modification of a freeway-to-freeway interchange; or
 - c. establishing a new partial interchange of any form.

2. FHWA Approval by Division Office. The Final IAD must be sent to the Division office for approval for the types of Interstate system new or revised access as follows:
 - a. establishing a new freeway-to-non-freeway (service) interchange;
 - b. modifying an existing freeway-to-non-freeway interchange configuration;
 - c. establishing locked-gate access; or
 - d. removing ramps or interchanges from service.

3. Time Limits of FHWA Approval. An affirmative determination by FHWA of safety, operational, and engineering acceptability for proposals for new or revised access points to the Interstate System should be reevaluated whenever a significant change in conditions occurs (e.g., land use, traffic volumes, roadway configuration or design, or environmental commitments). Proposals may be reevaluated if the project has not progressed to construction within 3 years of receiving an affirmative determination of engineering and operational acceptability (23 CFR 625.2(a); see also 23 CFR 771.129). If the project is not constructed within this time period, FHWA may evaluate whether an updated IAD based on current and projected future conditions is needed to receive either an affirmative determination of safety, operational, and engineering acceptability, or final approval if all other requirements have been satisfied (23 U.S.C. 111, 23 CFR 625.2(a), and 23 CFR 771.129). The NEPA document re-evaluation period is also 3 years (23 CFR 771.129).

48-1.04 Grade Separation Versus Interchange

Once it has been determined to provide a grade-separated crossing, the need for access between the two roadways with an interchange must be determined. The following lists several guidelines to consider when determining the need for an interchange:

1. Functional Classification. Interchanges should be provided at all freeway-to-freeway crossings. On fully access-controlled facilities, interchanges should be provided 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 which 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. When interchanges are spaced farther apart, freeway operations are improved. Spacing of urban interchanges between interchange crossroads should not be less than 1 mile. This should allow for adequate distance for an entering driver to adjust to the freeway environment, to allow for proper weaving maneuvers between entrance and exit ramps, and to allow for adequate advance and turnoff signing. In urban areas, a spacing of less than 1 mile may be developed by grade-separated ramps or by collector-distributor roads. In rural areas, interchanges should not be spaced less than 3 miles apart on the Interstate system or 2 miles on other systems.
4. Access. Interchanges may be required in areas where access availability from other sources is limited, and the freeway is the only facility that can practically serve the area.
5. Operations. Grade separated facilities without ramps will require all drivers desiring to turn onto the cross road to use other locations to make their desired moves. This will often improve the operations of the major facility by concentrating the turning movements at a few strategically placed locations. However, undue concentration of the turning movements at one location may overload the capacity of the exit or entrance facility.
6. Overpass vs. Underpass Roadways. A detailed study should be made at each proposed highway grade separation to determine whether the main road should be carried over or under the crossroad. Often the decision is based on features such as topography or functional classification.

48-2.0 INTERCHANGE TYPE SELECTION

The Traffic Engineering Corridor Development Office determines the interchange type for the site. Typically, the Corridor Development Office will evaluate several types for potential application.

48-2.01 General Evaluation

There are three overall factors that influence the selection of an interchange type:

1. Basic Type. 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. Urban/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. In urban areas where restricted right-of-way and close spacing of interchanges are common, the type selection and design of the interchange may be severely limited. The operational characteristics of the intersecting road and nearby interchanges will be major influences on the design of an urban interchange.
3. Movements. All interchanges should provide for all movements, even when the anticipated turning volume is low. An omitted maneuver may be a point of confusion to those drivers searching for the exit or entrance. In addition, unanticipated future developments may increase the demand for that maneuver.

Each interchange type should be evaluated considering:

- compatibility with the surrounding highway system;
- route continuity;
- level of service for each interchange element (e.g., freeway/ramp junction, ramp proper);
- operational characteristics (single versus double exits, weaving, signing);
- road user impacts (travel distance and time, safety, convenience and comfort);
- driver expectancy (e.g, exits and entrances to the right);
- geometric design;
- construction and maintenance costs;
- potential for stage construction;
- right-of-way impacts and availability;
- environmental impacts; and
- potential growth of surrounding area.

See Figure [48-2A](#) for general guidance for interchange types that are adaptable to freeways based on the functional classification of the intersecting facilities in rural, suburban or urban environments. At intersections other than a freeway-to-freeway, the choice of interchange will likely be limited to a cloverleaf, a diamond, or a variation thereof.

48-2.02 Interchange Types

In Indiana, there are six general interchange types: diamond, full cloverleaf, partial cloverleaf, three-leg, directional, and semi-directional. The following sections discuss these interchange types and the design elements for laying out the interchange, and additional information on alternate interchange designs to fit the site. The final design may be a minor or major modification of one of the general types or may be a combination of two or more general types.

48-2.02(01) Diamond Type Interchanges

The most prevalent type of interchange is the diamond. The diamond is a typical service interchange, which links a freeway with a lesser facility (arterial or collector surface street). Intersection control at ramp terminals is typically by way of signalization or signage. Variations of the diamond interchange include:

- Conventional Diamond
- Compressed Diamond (Tight Diamond)
- Diverging Diamond (DDI)
- Single Point Diamond Interchange (SPDI)
- Roundabout Diamond
- Three-Level Diamond

Additional guidance on Diamond Interchange type selection is available from JTRP SPR-3866 Guidelines for selecting Alternative Diamond Interchanges and Performance of Alternative Diamond Interchange Forms” by Purdue University; and HCM 6th Edition, Chapters 23 and 34 procedures for interchange type selection.

48-2.02(02) Full Cloverleaf

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 cloverleaf.

Where two access-controlled highways intersect, a full cloverleaf is the minimum type of interchange design that will suffice. However, a cloverleaf introduces several undesirable operational features such as the double exits and entrances from the mainline, the weaving between entering and exiting vehicles with the mainline traffic and, when compared to directional interchanges, the additional travel time and distance for left-turning vehicles. Therefore, a collector-distributor (C-D) road should be considered with a full cloverleaf, or a fully directional interchange should be provided. [See Figure 48-21](#) for typical examples of full cloverleaf interchanges with and without C-D roads. See Section 48-6.03 for a discussion on C-D roads.

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

1. Design Speed Impacts. For an increase in design speed, there will be an increase in:
 - travel distance,
 - required right-of-way, and
 - travel time
2. Loop Radii. Considering all factors, loops can be practically designed for approximate radii of 180 to 250-ft. The smaller radii are normally used in urban areas while the larger radii are typically used in rural areas.
3. Loop Geometry. Circular curve loop ramps are the most desirable geometrically because speeds and travel paths tend to be more constant and uniform. See Section 48-5.03 for ramp design horizontal alignment.
4. Loop Capacity. Expected design capacities for single-lane loops range from 800 to 1200 vph and, for 2-lane loops, 1000 to 2000 vph. The higher figures are generally only achievable where the design speed is 30 mph or higher and few trucks use the loop.

A loop ramp rarely operates with more than a single line of vehicles, regardless of the roadway width, and therefore, the cloverleaf capacity is limited by the loops. Loops may be made to operate with two lanes abreast, but only by careful attention to design of

terminals and the design for weaving, which would need widening by at least two additional lanes at the separation structure. To accomplish this type of design, the terminals should be separated by such great distances and the loop radii be made so large that cloverleafs with two-lane loops generally are not economical from the standpoint of right-of-way, construction, costs, and the amount of out-of-direction travel. Loops that operate with two lanes of traffic are more likely to be considered the exception as to the norm. Two-lane loop ramps should not be used where back to back loops are proposed.

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. When the total volume on the two successive ramps reaches approximately 1000 vph, interference increases rapidly with a resulting reduction of the through traffic speed. At these weaving volume levels, a collector-distributor road should be considered.
6. Weaving Lengths. The minimum weaving length between the exit and entrance gores of loops on new cloverleaf interchanges without C-D roads or those undergoing major reconstruction should be at least 1000-ft or the distance determined by a weaving analysis, whichever is greater.

Advantages of a full cloverleaf interchange include:

- Eliminates all vehicular stops through the use of merges.
- Eliminates all at-grade intersections and, therefore, eliminates left turns.
- Where right-of-way is reasonably inexpensive and adverse impacts are minimal, a full cloverleaf may be a practical option.

Disadvantages of a full cloverleaf interchange include:

- Requires more right-of-way and is more expensive than a diamond interchange.
- Loops operate at lower speeds. The loops in a cloverleaf result in a greater travel distance for left-turning vehicles (compared to diamond interchanges);
- Violates driver expectancy as half the exits and entrances are located beyond the crossroad structure;
- May introduce signing problems;
- Results in weaving sections. If the sum of traffic counts on two adjoining loops approaches 1,000 vehicles per hour, interference mounts rapidly, which results in a reduction of speed of through traffic. Consideration should be given to adding a collector-distributor road. The use of auxiliary lanes for acceleration or deceleration lanes is an alternative to collector-distributor roads.

- Generally, ramps at diamond interchanges can be easily widened to increase capacity; while, two-lane loop ramps which are rare, would require at least two additional lanes (one on each side) through the separation structure, longer weaving distances and may require a larger loop radius to operate.
- Pedestrian movements along cross streets can be difficult to safely accommodate at cloverleaf interchanges.
- A loop rarely operates with more than a single line of vehicles, and thus has a design capacity of 800 to 1,200 vehicles per hour.
- Potential for increased crash rates due to weaving issues.

48-2.02(03) Partial Cloverleaf

Partial cloverleaf (ParClo) interchanges are service interchanges with loops in one, two, or three quadrants. See [Figure 48-2J](#) for ParClo interchange schematics. Various configurations include:

1. ParClo-A Interchange. A ParClo-A has two-loop ramps are in opposite quadrants and accommodate the left turning vehicle from the cross street on to the freeway. The term ParClo-A refers to the location of the loop ramps in relation to the driver approaching the interchange. In the case of a ParClo-A, the loop on the driver's side of the approaching the interchange of the freeway is in advance of the cross street. The spacing of the intersections on the cross street are dependent on the radii of the loop ramps. The ParClo-A is constructed with two successive entrance spaced 1,000 to 1,500-ft apart in each direction of the freeway. Traffic from the two entrance ramps (loop ramp and right-turning ramp) may be merged first and then enter the freeway as a single entrance point where traffic volumes warrant or if volumes are small the two separate entrance design maybe used. Two phase signals would be utilized on the cross street intersections if warranted.
2. ParClo-B Interchange. This interchange type also has two loop ramps in opposite but different quadrants than the ParClo-A. For the driver on the freeway approaching the ParClo-B interchange, the loop ramp on the right side of the freeway is beyond the cross street.
3. ParClo-AB Interchange. Half the interchange functions as a ParClo-A and the other half functions as a ParClo-B. The design with loops in adjacent interchange quadrants causes weaving on the cross street between the loops. An auxiliary lane may be required between the loops to accommodate the weave.
4. ParClo-AB (Two Quad) Interchange. This interchange has all ramps developed on one side of the crossroad. Sometimes referred to as a “folded diamond”. This interchange is applicable where there are close parallel restrictions such as railroads, rivers, etc.

5. Single-Loop ParClo Interchange. This interchange is used when the volume of one of the left turning movements is extremely heavy. Adding a loop may not increase the capacity at an interchange under certain traffic volumes and/or conditions.

The above configurations 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.

Interchange ramps in only one quadrant have application for an intersection of roadways with low traffic volumes and minimal truck traffic. Where a grade separation is provided due to topography, and truck volumes don't justify the separation, a single two-way divided ramp of near minimum design usually will suffice.

Ramps should be arranged so that the entrance and exit movements create the least impediment to traffic flow on the major highway. The ramp arrangement should enable major turning movements to be made by right-turn exits and entrances.

Several of the advantages and disadvantages listed for full a cloverleaf also apply to a partial cloverleaf (e.g., geometric restriction of loops).

Specific advantages of a partial cloverleaf interchange include:

- May offer the opportunity to increase weaving distances, depending upon site conditions,.
- Often appropriate where one or more quadrants present adverse right-of-way and/or terrain problems.
- May reduce the number of left-turn movements when compared to a diamond interchange.
- ParClo-A and ParClo-B designs with loops in opposite quadrants eliminate the weaving problem associated with full cloverleaf design.

48-2.02(04) Three-Leg

Three-leg interchanges, also known as “T” or “Y” interchanges, are provided at intersections with three legs. [Figure 48-2K](#) illustrates examples of 3-leg interchanges with several methods of providing the turning movements. See the AASHTO *GDHS* for additional variations of the three-leg interchange. 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. Where

both left-turning movements are fairly heavy, the design in (B) or (C) may be suitable. A fully directional interchange (B) or (C) is appropriate when all turning volumes are heavy, or the intersection is between two access-controlled highways. These would be the most costly type because of the necessary multiple structures. A three-leg interchange should only be considered when future expansion in the unused quadrant is either impossible or highly unlikely. They are very difficult to expand or modify in the future. See Section 48-4.05 for applicable major divergence and branch connection design.

48-2.02(05) Directional and Semi-Directional

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

1. Directional Ramp. A ramp that does not deviate greatly from the intended direction of travel. See [Figure 48-2L](#).
2. Semi-Directional Ramp. A ramp that is indirect in alignment, yet more direct than loops. See [Figure 48-2M](#).
3. Fully Directional Interchange. An interchange where all left-turn and right-turn movements are provided by directional ramps. See [Figure 48-2L](#).
4. 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 48-2M](#).

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 cloverleaf interchanges. Left-hand exits and entrances violate driver expectancy and, therefore, should be avoided.

Directional or semi-directional interchanges are most often warranted in urban areas at freeway-to-freeway or freeway-to-arterial intersections. They may 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. Interchanges involving two freeways will almost always require directional layouts. See Section 48-4.05 for applicable major divergence and branch connection design.

48-3.0 TRAFFIC OPERATIONAL FACTORS

48-3.01 Basic Number of Lanes

The basic number of lanes is the minimum number of lanes designated and maintained over a significant length of a route based on the overall operational needs of that section. The number of lanes should remain constant over significant distances. For example, a lane should not be dropped at the exit of a diamond interchange and then added at the downstream entrance simply because the traffic volume between the exit and entrance drops significantly. Likewise, a basic lane should not be dropped between closely spaced interchanges simply because the estimated traffic volume in that short section of highway does not warrant the higher number of lanes.

48-3.02 Lane Balance

Lane balance refers to certain principles which apply at freeway exits and entrances as follows:

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 exits which follow a loop ramp entrance or at exits between closely spaced interchanges (i.e., interchanges where the distance between the end of the taper of the entrance terminal and the beginning of the taper of the exit terminal is less than 1500-ft and a continuous auxiliary lane between the terminals is being used). In these cases, the auxiliary lane may be dropped in a single-lane exit with the number of lanes on the approach roadway being equal to the number of through lanes beyond the exit plus the lane on the exit. [Figure 48-4G](#) illustrates a taper type multi-lane exit ramp design. The configuration provides lane balance and increased weaving capacity when a continuous auxiliary lane is present due to its option lane feature.
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. It may be equal to the number of traffic lanes on the merging roadway.
3. Traveled Way. The traveled way width of the highway should not be reduced by more than one traffic lane at a time.

The following violate the principle of lane balance:

- dropping two lanes at a 2-lane exit ramp. One lane should provide the option of remaining on the freeway.
- immediately merging both lanes of a 2-lane entrance ramp into a highway mainline without adding at least one additional lane beyond the entrance ramp.

- immediately adding two or more lanes at the same location on a freeway, even if it is in advance of a multi-lane exit ramp. An auxiliary lane (preferably 1500 ft min.) should be included between each successive added lane or each lane reduction.

[Figure 48-3A](#) illustrates how to achieve lane balance at the merging and diverging points of branch connections.

48-3.03 Weaving Analysis

A freeway facility is comprised of three types of segments: weaving segment, ramp junctions and basic freeway segments. While these segments have different operating characteristics, conditions within particular segment impact the traffic flow conditions of upstream and downstream segments.

Weaving is generally defined as the crossing of two or more traffic streams traveling in the same general direction along a significant length of highway without the aid of traffic control devices (except for guide signs). Thus, weaving sections are formed when merge areas are closely followed by diverge areas. The term “closely” implies that there is not sufficient distance between the ramp merge and diverge areas for them to operate independently. The AASHTO *GDHS* recommends a minimum ramp spacing of 2000 feet between a system and a service interchange and a minimum of 1600 feet between two service interchanges. See [Figure 48-3B](#). The minimum ramp terminal spacing is independent of design speed. When minimum spacing cannot be met, spacing which accommodates the AASHTO *GDHS* Decision Sight Distance for Avoidance Maneuver *C* and *E* should be considered.

Weaving sections require intense lane-changing maneuvers as drivers must access lanes appropriate to their desired exit points. Traffic in a weaving section is, therefore, subject to lane changing turbulence in excess of that normally present on basic freeway sections.

48-3.03(01) Weaving Analysis Characteristics

There are three geometric characteristics of weaving sections that affect its operating characteristics:

- length,
- width
- configuration

Length is the distance between the merge and diverge areas forming the weaving section. *Width* refers to the number of lanes within the weaving section. *Configuration* is defined by the way entry

and exit lanes are aligned with each other. All of these characteristics have an impact upon the critical lane changing activity of a weaving section.

[Figure 48-3C](#) illustrates two ways in which the length of a weaving section may be reasonably measured. These lengths correspond to the 2010 Highway Capacity Manual (HCM) and are defined as follows:

L_S = Short Length; the distance between the end points of any pavement markings that prohibit or discourage lane-changing.

L_B = Base Length; the distance between points in the respective gore areas where the left edge of the ramp travel lanes and the right edge of the freeway travel lanes meet.

Previous versions of the HCM tied weaving length to the specifics of the loop ramp design in a cloverleaf interchange as most weaving sections were part of such interchanges. Modern weaving sections cover a wide range of designs and situations, and a more general definition of length is, therefore, appropriate.

Type A, B and C weaving configurations as defined in the 2000 HCM have been redefined into two categories as either one-sided or two-sided weaving sections.

- One-sided Weaving Section. A one-sided weaving section is a weaving section in which no weaving maneuver requires more than two lane changes.
- Two-sided Weaving Section. A two-sided weaving section is a weaving section formed by a single-lane on-ramp followed closely by a single-lane off-ramp where the ramps are on opposite sides of the freeway; or any weaving section in which one weaving movement requires three or more lane change.

Most weaving sections are of the one-sided variety. In general, this means that the ramps that define the entry to and exit from the weaving section are on the same side of the freeway – either both on the right (most common) or both on the left.

See [Figure 48-3C](#) for illustrations of one-sided and two-sided weaving sections.

- Illustration (A) shows a typical one-sided weaving section formed by a one-lane, right side on-ramp followed closely by a one-lane, right-side off-ramp, connected by a continuous auxiliary lane. Each weaving vehicle must make one lane change, as illustrated, and the lane changing turbulence caused is clearly focused on the right side of the freeway.
- Illustration (B) a typical *major weaving section*. A major weave is formed when one or more entry/exit legs have multiple lanes. This is considered as a one-sided weaving section

in which the on-ramp has two lanes. One weaving movement can be made without a lane change (freeway to ramp), while the other (ramp to freeway) requires one lane change.

- Illustrations (C) shows the most common form of two-sided weave scenarios in which a one-lane on-ramp on one side of the freeway (in this case, on the left) is followed closely by a one-lane off-ramp on the other side of the freeway (in this case, on the right). Even though the ramp-to-ramp weaving movement makes only two lane changes, this is still classified as two-sided weaving.
- Illustration (D) shows the less common case in which one of the ramps has multiple lanes. The ramp-to-ramp weaving movement must execute three lane changes.

48-3.03(02) Weaving Analysis Level of Service (LOS)

The LOS of the weave should be at least equal to the mainline segment LOS but not lower than one LOS below that of the mainline segment.

Addition information on weave analysis is available in the HCM.

48-3.04 Route Continuity

All highways with interchanges are designated by route number. Desirably, the through driver should be provided a continuous numbered route on which changing lanes is not necessary to continue on the through 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, but rather, the through route.

48-3.05 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. The Traffic Engineering Division will determine the use of traffic control devices at interchanges.

48-3.06 Uniformity

To the extent practical, all interchanges along a freeway should be reasonably uniform in geometric layout and appearance. Except for highly specialized situations, all entrance and exit ramps should be to the right.

48-3.07 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 junctions should provide for vehicular maneuvering, signing and capacity. The ramp-pair combinations are entrance followed by entrance (EN-EN), exit followed by exit (EX-EX), exit followed by entrance (EX-EN), entrance followed by exit (EN-EX). The final decision on the 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*. [Figure 48-3B](#) shows the recommended minimum ramp terminal spacing.

48-3.08 Auxiliary Lanes

As applied to interchange design, auxiliary lanes are most often used to comply with the principle of lane balance to accommodate speed change, increase capacity, and weaving for entering and exiting vehicles. An auxiliary lane may be dropped at an exit if properly signed and designed. The following statements apply to the use of an auxiliary lane within or near interchanges:

1. Within Interchange. [Figure 48-3D](#) provides the basic schematics of alternative designs for adding and dropping auxiliary lanes within interchanges. The selected design will depend upon traffic volumes for the exiting, entering and through movements.

The distance between the end of the entrance taper (without the connecting auxiliary lane) and the beginning of the downstream exit taper is relatively short (e.g., 1500-ft or less), and/or

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 to form a continuous auxiliary lane.

An existing auxiliary lane can be retrofit between an entrance and an exit ramp by extending the auxiliary lane beyond the physical nose of the exit ramp gore to

accommodate merging traffic. The exit gore should be visible throughout the length of the auxiliary lane. See [Figure 48-3D\(c\)](#).

Design details for exits and entrances are provided in Section 48-4.0, and design details for lane drops are provided in Section 48-6.02.

48-3.09 Safety Considerations

Safety is an important consideration in the selection and design of an interchange. After many years of operating experience and safety evaluations, certain practices are considered less desirable at interchanges nationwide. The following summarizes several major safety considerations.

1. Exit Points. Many interchanges have been built with exit points which could not clearly be seen by approaching drivers. Decision sight distance should be provided where practical at freeway exits. There should be a clear view of the entire exit terminal, including the exit nose. See Section 48-4.01 for the application of decision sight distance to freeway exits.

Where traffic warrants for cloverleaf and various configurations of ParClo interchanges, a C-D roadway system may be considered where multiple exit and/or entrance ramps may be combined into a single exit/entrance point. Proper advance signing of exits is essential.

2. Exit Speed Changes. Freeway exits should provide sufficient distance for a safe deceleration from the freeway design speed to the design speed of the first governing geometric feature on the ramp, typically a horizontal curve.
3. Merges. Rear-end collisions on entrance merges onto a freeway may result from a driver attempting the complicated maneuver of simultaneously searching for a gap in the mainline traffic stream and watching for vehicles in front. An acceleration distance of sufficient length should be provided to allow a merging vehicle to attain speed and find a sufficient gap to merge into.
4. Driver Expectancy. Interchanges can be significant sources of driver confusion; therefore, they should be designed to conform to the principles of driver expectation. Left-hand merges are not desirable. It is difficult for a driver entering from a ramp to safely merge with the high-speed left lane on the mainline. Therefore, left exits and entrances should not be used, because they are not consistent with the concept of driver expectancy when they are mixed with right-hand entrances and exits. In addition, exits should not be placed in line with the freeway tangent section at the point of mainline curvature to the left.

5. Fixed Objects. Because of traffic operations at interchanges, a number of fixed objects may be located within interchanges, such as signs at exit gores or bridge piers and rails. These should be removed, where practical, made breakaway or shielded with barriers or crash cushions. Horizontal stopping sight distance should be considered. With the minimum radius for a given design speed, the normal lateral clearance at piers and abutments of underpasses does not usually provide the minimum stopping sight distance. Thus, above-minimum radii should be used for curvature on highways through interchanges. See Chapter 49.
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, poor channelization or inadequate signing. The interchange design must attempt to minimize wrong-way possibilities.
7. Weaving. Areas of vehicular weaving may create a high demand on driver skills and attentiveness. Where practical, interchanges should be designed without weaving areas or, as an alternative, with weaving areas removed from the highway mainline (e.g., with collector-distributor roads).
8. Crossroad. The crossroad at a rural freeway interchange should be a divided roadway through the interchange area.

48-3.10 Capacity and Level of Service

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

1. Basic freeway section where interchanges are not present,
2. Freeway-ramp terminals,
3. Weaving segments,
4. Ramp proper,
5. Ramp/crossroad intersections, and
6. Collector-distributor roadways.

The *Highway Capacity Manual* (HCM) provide techniques for analyzing the capacity and level of service (LOS) for each element listed above. Highway Capacity Software is required for the analysis for mainline interstate, weaving segments, and ramp junction analysis. Use of other transportation analytical software must be compliant with the applicable sections of the HCM and is subject to approval by the Corridor Development Office.

The interchange should operate at an acceptable LOS. The LOS values presented in Tables 53-1 and 54-2A for freeways will also apply to interchanges. The LOS of each interchange element should be as good as the LOS provided on the basic freeway section. Interchange elements should not operate at more than one LOS 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 mainline. This will likely involve a consideration of the operational characteristics on the minor road for some distance in either direction from the interchange. For State projects, the Corridor Development Office is responsible for conducting the preliminary capacity analyses at interchanges.

48-3.11 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. The designer should have available the peak-hour volumes, number of traffic lanes, etc., to determine the type of traffic the driver will encounter.

48-4.0 FREEWAY/RAMP JUNCTIONS

48-4.01 Exit Ramps

48-4.01(01) Types of Exit Ramps

There are two basic types of exit freeway/ramp junctions - the parallel design and the tapered design. For all new and reconstructed service interchange exit ramps, INDOT's preferred practice is to use the parallel design for single-lane ramps.

The use of a tapered design requires the approval of the Highway Design and Technical Support Director. Ramp design speeds must be in the middle to upper range of mainline design speed. Existing tapered exit ramp designs may be retained if deemed operationally acceptable and there is not an adverse crash history at the ramp junction.

The designer may consider replacing an existing single-lane taper design with a parallel design where:

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

2. a taper design cannot be improved to provide the necessary deceleration distance prior to a sharp curve on the ramp;
3. the exit ramp departs from a horizontal curve on the mainline. The parallel design is less confusing to through traffic and will normally result in smoother operation;
4. the need is satisfied for a continuous auxiliary lane. See Section 48-3.08. If the exiting volume warrants a multi-lane exit ramp, a taper type multi-lane exit may be considered to satisfy weaving capacity and to provide lane balance;
5. the capacity of the at-grade ramp terminal is insufficient and ramp traffic may back up onto the freeway.

See [Figures 48-4A](#) and [48-4B](#) for detailed design information for INDOT's typical single lane, parallel and tapered exit freeway/ramp junctions (service interchanges). See Figures 48-4F and 48-4G for detailed design information for INDOT's typical multi-lane, parallel and tapered exit freeway/ramp junctions (service interchanges).

For system interchanges, see Section 48-4.04, Major Forks/Branch Connections.

48-4.01(02) Taper Rates

For a parallel-lane exit design, the taper rate applies to the beginning taper of the parallel lane. This distance is typically 300-ft as illustrated in [Figure 48-4A](#).

48-4.01(03) Divergence Angle

The divergence angle is the angle of departure from the mainline traveled way on a freeway to the exit ramp which typically develops a taper along the gore at a rate which ranges from 20:1 to 30:1. The AASHTO *GDHS* allows the divergence angle to range from 2° to 5°. To provide uniformity through-out Indiana's highway system, all new and reconstructed service interchange single and multi-lane exit ramps should have a divergence angle of 2°17'26" (25:1 taper) as shown in [Figure 48-4A](#). Exceptions to this practice require approval from the Highway Design and Technical Support Division Director.

48-4.01(04) Deceleration

Sufficient deceleration distance is needed to safely and comfortably allow a vehicle to exit the freeway mainline. For a parallel design (Figure 48-4A), the minimum deceleration length is the minimum length including adjustments for grade, or 800 ft, whichever is greater. See Figures [48-4K](#) and [48-4I](#) for minimum deceleration lengths and adjustment values, respectively. Additional length beyond 800 ft should be added to the parallel segment located adjacent to the 300-ft taper. In restrictive area where it is impractical to extend the parallel segment, the deceleration length may include the first curve downstream from the gore nose, provided the curve radius exceeds 3000 ft and the curve length exceeds 300 ft.

Where a tapered design exit ramp is approved for use, the ramp design speed must be in the middle to upper range of the mainline design speed shown in [Figure 48-5A](#). For a tapered design, the minimum deceleration length is 600 ft. This length encompasses downgrades up to 6% and ramp design speeds up to 50 mph.

48-4.01(05) Sight Distance

The sight distance approaching the gore nose should exceed the stopping sight distance for the through traffic, desirably by 25% or more. Where there are unusual conditions, consider providing decision sight distance to the exit terminal. Extra sight distance is particularly important for exit loops immediately beyond a structure. When measuring for adequate sight distance, ensure that the motorist can see the pavement surface at and beyond the gore nose. Locating the exit terminal and gore nose where the mainline is on an upgrade provides the best design condition. Do not locate exit terminals near mainline crest vertical curves where the ramp pavement may disappear from the driver's view.

48-4.01(06) Superelevation [Rev. Mar. 2025]

Superelevation for horizontal curves near the mainline/ramp junction must be developed to properly transition the driver from the mainline to the curvature at the exit. The principles of superelevation for open-roadway conditions, as discussed in Chapter 305, should be applied to the mainline/ramp junction. If drainage impacts to adjacent property or frequency of slow-moving vehicles are important considerations, low speed urban criteria may be used if the design speed on the ramp proper is 45 mph or less. The following will apply to superelevation development at exit ramps:

1. e_{\max} . On the exit ramp portion of the mainline/ramp junction, the typical e_{\max} is 6% or 8%. See Section 305-4.0.

2. Superelevation Rate. As discussed in Chapter 305, Method 5 is used for open-roadway conditions to distribute superelevation and side friction. The designer will use the ramp design speed and the curve radius to determine “e”, subject to R_{min} for the ramp design speed. The superelevation rate and radii used should reflect a decreasing sequence of design speeds for the exit terminal, ramp proper, and at-grade terminal for diamond ramps.
3. Transition Length. The designer should use the superelevation transition lengths for 2-lane roadways as presented in Section 305-4.04 to transition the exit ramp cross slope to the superelevation rate at the PC.
4. Distribution. Depending on the number of lanes rotated and design speed, the superelevation transition length should be distributed such that 67% ($2/3^{rds}$) of the length is in advance of the PC and the remaining 33% ($1/3^{rd}$) is beyond the PC. However, at freeway/ramp junctions, field conditions may make this distribution impractical, and a different distribution may be necessary. However, it should not be less than 50/50.
5. Axis of Rotation. This will typically be about the centerline of the ramp travelway for two-lane ramps and may be about either the centerline or the outside edge of travelway for single-lane ramps.

48-4.01(07) Cross Slope Rollover

The cross-slope rollover is the algebraic difference between the slope of the through lane and the slope of the entrance lane, where these two are adjacent to each other. At freeway entrances and exits, the maximum algebraic difference between adjacent lanes and gore areas should not exceed 5%.

See Section 48-4.01(09) for nose definitions.

48-4.01(08) Shoulder Transition

The right shoulder of the mainline will be transitioned to the narrower shoulder of the ramp. As illustrated in [Figures 48-4A](#) and [48-4B](#), the shoulder width along the mainline will be maintained until 100-ft before the gore nose or ramp PC. The shoulder width will then be transitioned to the ramp right shoulder width (typically 8 ft). In restricted areas, it is acceptable to provide a 6-ft minimum right shoulder along the entire parallel exit ramp area.

48-4.01(09) Exit Gore Area

The term *gore* indicates an area downstream from the intersection point of the mainline and exit shoulders. 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 300 ft downstream beyond the gore nose.

See [Figure 48-4D](#) for exit and entrance ramp gore details. The following definitions will apply:

1. Painted Nose. The painted nose, also called the theoretical gore, 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 travelway.
2. Dimension Nose. The dimensional nose is a point where the shoulder is considered to begin within the gore area. For exit ramps, the dimension nose is 4-ft wide.
3. Physical Nose. The physical nose is the point where the ramp and mainline shoulders converge. The physical nose has a dimensional width of 14-ft.
4. Gore Nose. The gore nose is the point where the paved shoulder ends and the sodded area begins as the ramp and mainline diverge from one another. The gore nose has a dimensioned width of 6-ft and does not include the shoulders. The total width of the gore nose including the shoulders should be a minimum 20 ft.

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 beyond the gore nose. Any obstacles within 300 ft of the gore nose are to be made breakaway or shielded by a barrier. See Section 49-3.0.
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-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 Section 49-3.0).
3. Cross Slopes. The paved triangular gore area between the through lane and ramp should be traversable. The cross slope is the same as that of the mainline (typically 2%) from the painted nose up to the dimension nose. Beyond this point, the gore area may be depressed to direct drainage to an inlet or discharge point with cross slopes of 2-4%. See Section 48-4.01(07) and [Figure 48-4D](#) 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 Section 502-1.0. See Section 502-2.0 for the pavement marking details in the triangular area upstream from the gore nose.
5. Drainage. Positive drainage within the gore area should be provided by either directing flow to an inlet or by sheet flow to the outside across the ramp. Slotted drains are prone to clogging, may affect traversability, and should not be used in the gore. See [Figure 48-4D](#) for cross information relative the ramp and adjacent pavement.

48-4.02 Entrance Ramps

48-4.02(01) Types of Entrance Ramps

There are two basic types of entrance freeway/ramp junctions – the parallel design and the tapered design. [Figure 48-4C](#) includes detailed design information for these two entrance freeway/ramp junctions. It is INDOT preferred practice to only use the parallel design on new and reconstructed ramps for single and multi-lane entrance ramps. The parallel design offers several advantages when compared to the taper design, including:

1. Where the LOS 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. 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. 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.
4. 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.

48-4.02(02) Merge Taper Rates

For parallel design entrance ramps, the taper at the merge point is 600-ft minimum (50:1). For ramps with high truck volumes, use a 70:1 taper.

48-4.02(03) Acceleration

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

1. Passenger Cars. [Figure 48-4H](#) provides the minimum lengths of acceleration for passenger cars. For both parallel and tapered designs, a portion of the ramp proper may be included in the acceleration lane length where the curve approaching the acceleration lane has a radius equal to or greater than 1000 ft. Parallel ramps must also have a minimum length of curve of at least 200 ft. INDOT's standard ramp configuration for both parallel and tapered entrance ramps uses a minimum radius of 3819.72 ft (3 degrees) for the approaching curve with a curve length of 200-ft. The acceleration distance is measured from the point of the last controlling curve to the beginning of the merge taper (see [Figure 48-4C](#)). Where upgrades exceed 3% over the acceleration distance, the acceleration length should be adjusted according to the values presented in [Figure 48-4I](#).

INDOT's acceleration lengths provide sufficient distance for 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 desirably total 1200 ft, exclusive of the taper, to provide additional merging opportunities.

2. Trucks. Where the existing or forecasted truck volumes are $\geq 10\%$ of AADT or 20 trucks per hour, the Traffic Engineering Division of Corridor Development should be contacted to determine if the truck acceleration distances provided in [Figure 48-4J](#) are to be considered in the ramp design. Only parallel entrance ramps designs are acceptable at locations where truck traffic dictates the design. Typical areas where trucks might govern the ramp design will include weigh stations, 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. the junction operates at a LOS D or worse,
 - b. a significant accident history involving trucks which can be attributed to an inadequate acceleration length, and/or
 - c. an undesirable level of vehicular delay at the junction attributed to an inadequate acceleration length.

Where upgrades exceed 2%, the truck acceleration distances may be corrected for grades. Figures 44-2B and 44-2C provide performance criteria for trucks on accelerating grades. Before providing any additional acceleration lane 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 Section 305-3.0
 - 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.

48-4.02(04) Sight Distance

Decision sight distance should desirably be provided for drivers on the mainline approaching an entrance terminal. They need 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. Section 42-2.0 discusses decision sight distance in more detail.

48-4.02(05) 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 Section 305-4.01, should be applied to the entrance design. Section 48-4.01(06) 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, distribution of transition lengths between curve and tangent, and the axis of rotation.

48-4.02(06) Cross Slope Rollover

The cross slope rollover is the algebraic difference between the slope of the through lane and the slope of the entrance lane, where these two are adjacent to each other. At freeway entrances and exits, the maximum algebraic difference between adjacent lanes and gore areas should not exceed 5%.

48-4.02(07) Shoulder Transitions

At entrance terminals, the right shoulder must be transitioned from the narrower ramp shoulder to the wider freeway shoulder. See Figure [48-4C](#) for typical shoulder transition details. In restricted areas, it is acceptable to maintain the 6-ft right shoulder width on the ramp throughout the parallel lane until the beginning of the merge taper with the mainline.

48-4.02(08) Entrance Gore Area

Section 48-4.01(09) provides general design considerations for exit gores and the definitions for various nose types which are within the gore area. The following presents the nose dimensions for entrance gores. See [Figure 48-4D](#) for entrance and exit ramp gore details.

1. Painted Nose. Also called the theoretical gore, 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 travelway.
2. Dimension Nose. The dimension nose width for entrance ramps is 2 ft.
3. Physical Nose. The physical nose has a dimensioned width of 14 ft.
4. Gore Nose. The gore nose has a dimensioned width of 6 ft and does not include the shoulders. The total width of the gore nose including the shoulders should be at a minimum of 20 ft.

48-4.03 Multi-Lane Terminals

Multi-lane terminals may be considered when the capacity of the ramp is too great for single-lane operation. Typically multi-lane ramps are more effective at improving capacity for off-ramps than on-ramps. In both cases, the overall increase in capacity is contingent upon downstream conditions.

The design should consider the following elements for a multi-lane terminal:

1. Two-Lane Off-Ramp. A multi-lane off-ramp typically provides more capacity than a single-lane ramp as flow increases through the diverge area. However, the diverge area is controlled by the capacity of the exiting roadway.
2. Two-Lane On-Ramp. A multi-lane on-ramp will typically achieve a merge with less turbulence and a higher LOS but will not increase the capacity of the merge, which is controlled by the downstream freeway segment. Although the capacity of a two-lane on ramp is approximately double that of a single lane ramp, it is unlikely that a two-lane ramp can accommodate more than 2250 to 2400 passenger cars per hour through the merge itself. Longer acceleration lanes associated with two-lane ramps results in less turbulence as ramp vehicles enter the freeway traffic stream. This leads to lower densities on the influence area and higher flows in the ramp lanes.
3. Lane Balance. Lane balance at the freeway/ramp junction should be maintained. See Section 48-3.02.
4. Loop Ramps. Where the capacity analysis indicates that a single-lane loop capacity is insufficient, consideration should be given to providing either a 2-loop ramp or a direct connection ramp. For 2-lane loop ramps, the designer should consider the following:
 - a. Two-lane loop ramps should have a minimum radius of 200 ft (180 ft for restrictive conditions). The loop travel-way should not be less than 30 ft (2 15-ft lanes) and for radii less than 200 ft use a travelway width of 32 ft.
 - b. Expected design capacities for single-lane loops range from 800 to 1200 vph and for 2-lane loops, 1000 to 2000 vph.
 - c. Enough distance needs to be provided to properly design the exit and entrance for the second lane on the loop.
5. Entrances. INDOT's preferred practice for multi-lane entrance ramps is to use a parallel design. See [Figure 48-4E](#) for parallel multi-lane entrance ramp design details.
6. Exits. Multi-lane exit ramps may be either a parallel design or a tapered design with an option lane.
 - a. Parallel Multi-lane Exit Ramp. For a parallel multi-lane exit ramp, the first auxiliary lane should be added at least 1500 ft in advance of the exit taper. The total length from the beginning of the first taper to the gore nose should range from 3100 ft for turning volumes of 1500 vph or less upward to 4100 ft for turning volumes of up to 3000 vph. See [Figure 48-4F](#) for parallel multi-lane exit ramp design details.

Where a ramp splits or forks beyond the painted nose of the exit ramp, two parallel deceleration lanes should be provided prior to the gore nose for the 2500-ft length mentioned above. The exit taper to the parallel deceleration lanes should be a minimum of 300 ft long. This parallel design should also be considered where vehicle storage is anticipated in the ramp lanes and deceleration lanes in advance of the crossroad intersection.

- b. **Tapered Multi-lane Exit Ramp with Option Lane.** The tapered design multi-lane exit ramp design has an option lane that allows a driver to remain on the outside lane of the mainline or to exit onto the inside lane of the exit ramp without a lane change. The option lane feature reduces the number of lane changes and automatically provides a lane-balanced exit. The option lane configuration requires providing an additional auxiliary lane of at least 2000 ft. The total length from the beginning of the first taper to the gore nose should be a minimum of 3100 ft and has the capacity to accommodate turning volumes up to 3000 vph. See [Figure 48-4G](#) for tapered multi-lane exit ramp design details.

The tapered multi-lane exit ramp may offer advantages to the parallel multi-lane exit ramp design as follows:

- The tapered multi-lane exit ramp design may be more economical due to its reduced footprint.
- Environmental impacts and right-of-way acquisition are typically lessened due its reduced footprint.
- The option lane feature reduces lane weaves and increases weaving capacity.

7. **Signing.** The geometric layout of multi-lane exits must be coordinated with the Traffic Division Office of Traffic Design because of the complicated signing which may be required in advance of the exit.

48-4.04 Major Fork/Branch Connections

Major fork (divergence) and branch connections (convergence) are used for system interchanges. See Figures [48-4L](#) and [48-4M](#) for typical design details for a major forks and branch connections, respectively.

The designer should consider the following when designing major fork and branch connections:

1. **Lane Balance.** The principle of lane balance should be maintained. See Section 48-3.02.

2. Divergence Point. Where the alignments of both roadways are on horizontal curves at a major fork, the painted nose of the gore should be placed in direct alignment with the centerline of one of the interior lanes. This provides a driver in the center lane the option of going in either direction. See Figure 48-4L, schematics A, B and C. Where one of the roadways is on a tangent at a major fork, the gore design should be similar to freeway/ramp multi-lane exit. See Figure 48-4L, schematic D.
3. Nose Width. At the painted nose of a major fork, the lane should be at least 24-ft wide but preferably not over 28 ft. The widening from 12 ft to 24 ft should occur within a distance of 1000 ft to 1800 ft. See Figure 48-4L, schematic A.

If a design hourly volume of greater than 1500 vph is anticipated on the exit ramp at a major fork on a systems interchange, the exit deceleration lanes, exclusive of the exit tapers, should begin approximately 1 mile before the painted gore nose, but not less than 2700 ft.

4. Branch Connection. A branch connection forms when two separate multilane freeway ramps or routes converge to form a single freeway route.

Traffic demand may indicate that the number of lanes beyond the convergence point should equal the combined total number of lanes on the two approach roadways. See Figure 48-4M, schematic A. Otherwise, the number of lanes downstream from the point of convergence may be one less than the combined total on the two approach roadways. When merging, a full lane width should be carried for at least 1000 ft beyond the painted nose and tapered at a minimum of 50:1, preferably at 70:1. See Figure 48-4M, schematic B.

There are no effective models of performance for a major merge area (*HCM v6.0, Chapter 14*). Therefore, analysis is limited to checking capacities of the approaching legs and the departing freeway. Problems in major merge areas generally result from insufficient capacity of the downstream freeway segment.

Lane drops should be in accordance with Section 48-6.02, Freeway Lane Drops. Because the outer lane from the roadway entering from the left is the low speed lane for that roadway and the inside lane from the roadway entering from the right is the high speed lane for that roadway, turbulence is likely at the convergence point. Consideration should be given to providing more than the 1000-ft minimum to alleviate the turbulence to the extent practical.

48-5.0 RAMP DESIGN

48-5.01 Design Speed

The ramp design speeds may vary. The designer should use the acceptable ranges listed for the ramp types listed below and Figure [48-5A](#) to determine the ramp design speed based on the design speed of the mainline:

1. Freeway/Ramp Junctions. The design speeds in Figure [48-5A](#) 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 the figure may not be applicable to the ramp portion near the intersection. For additional information on the design speed selection near at-grade intersections, see Chapter 46.
3. Variable Speeds. The ramp design speed may vary based on the two design speeds of the intersecting roadways. Higher design speeds should be used on the portion of the ramp near the higher-speed facility while lower speeds may be selected near the lower-speed facility. When using variable design speeds, the maximum speed differential between controlling design elements (e.g., horizontal curves, reverse curves) should not be greater than 10 to 20 mph. The designer needs to ensure that sufficient deceleration distance is available between design elements with varying design speeds (e.g., two horizontal curves).
4. Ramps for Right Turns. Design speeds for right-turn ramps are typically in the mid- to high range. This includes, for example, a diagonal ramp of a diamond interchange.
5. Loop Ramps. Design speeds in the high range are generally not attainable for loop ramps. Minimum values usually control. For mainline design speeds greater than 50 mph, the loop design speed should not be less than 20 mph. However, design speeds greater than 30 mph will require significantly more right-of-way and may not be practical in urban areas. Normally, a loop should not be designed for a speed greater than 35 mph. Arterial loop ramp radii should desirably be greater than 150 ft.
6. Semi-direct Connections. Design speeds between the mid- to high ranges should be used for semi-direct connections. Design speeds greater than 50 mph are generally not practical for short, single-lane ramps. For 2-lane ramps, values in the mid- to high ranges should be used.
7. Direct Connections. For direct connections, the design speed should be in the mid to high range.

48-5.02 Cross Section [Rev. Feb. 2019]

See Figure [48-5B](#) for single and Figure [48-5C](#) for multilane ramp typical cross sections for tangent and for superelevated scenarios. The following will also apply to the ramp cross sections:

1. Width. The minimum paved width of a 1-way, 1-lane ramp will be 28 ft. The 28-ft width includes a 4-ft left shoulder, an 8-ft right shoulder and a 16-ft travelway. Multi-lane ramp widths should be in multiples of 12 ft, with a 4-ft wide left shoulder and a 10-ft wide right shoulder. The guardrail offset from the edge of shoulder should be 2 ft. The bridge railing offset should be 1'-8". Full-depth paving equal to the ramp pavement thickness should be provided on the shoulders because of frequent use of shoulders for turning movements and passing stalled vehicles

The designer must request approval from the Department to reverse the left and right shoulder widths to provide additional sight distance for ramps that have tight or prolonged curves to the left.

2. Pavement Design. Loop ramps and other ramps with curve radii less than or equal to 300-ft should be designed with full-depth pavement for the entire 28-ft width. For ramps with curve radii greater than 300-ft, only the 16-ft traveled way will typically have a full-depth pavement structure. Outer connector ramps at a cloverleaf interchange or the ramps at a diamond interchange should have full-depth shoulders. For additional pavement design information, see Chapter 304.
3. Cross Slope. On a tangent section of a single lane ramp, the cross slope of the traveled way and the left shoulder match, typically at 2%. The right shoulder cross slope is typically 4%. For all superelevated ramps, the entire ramp width, including the shoulders, should have the same cross slope. The cross slopes of multi-lane ramps are the same as the cross sectional elements of the freeway mainline typical tangent section. See Figure 48-5B for single lane sections and Figure 48-5C for multi-lane sections.
4. Curbs. In general, curbs should not be used on ramps. However, mountable curb may be used for drainage or to prevent erosion on steep embankment slopes. See Section 49-3.04 for additional curbing information. Curbs may be placed at the edge of the roadway of a ramp on a low speed facility if approved by the Department.
5. Bridges and Underpasses. The full paved width of the ramp should be carried 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. See Section 45-3.0 and Section 45-8.0 for additional information on 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 Figure 49-2A. The design ADT will be the directional ADT on the ramp.
8. Barriers. Whenever practical, an additional 2-ft should be added to the shoulder width when a roadside barrier is used. Where a barrier is present on a horizontal curve, the designer should determine the barrier impact on horizontal sight distance. See Section 305-5.0.
9. Right-of-Way. The right-of-way adjacent to the ramp should be limited access right-of-way.

48-5.03 Horizontal Alignment

48-5.03(01) Theoretical Basis [Rev. Mar. 2025]

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 305 discusses the theoretical basis for horizontal alignment assuming open-road conditions. In summary, this includes the following:
 - 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
 - d. for a simple curve, depending on the number of lanes rotated, superelevation runoff length may be distributed using the $2/3^{\text{rd}}$ – $1/3^{\text{rd}}$ rule, which places $2/3^{\text{rd}}$ on the tangent and the remaining $1/3^{\text{rd}}$ on the horizontal curve. A spiral curve transition may be considered where there is a steep downgrade with high volumes and a sharp curve with maximum superelevation. Coordination with the Traffic Division Corridor Development Office is required prior to utilizing a spiral curve transition. See the AASHTO *GDHS* for additional design guidance on spiral curves.

2. Turning Roadway Conditions: Section 46-3.02 discusses the theoretical basis for horizontal alignment assuming turning roadway conditions. In summary, this includes the following:
 - a. higher side-friction factors than open-road 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.
 - 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, including:

- freeway/ramp junction
- ramp proper (directional ramps)
- ramp proper (loop ramps)
- ramp terminus (intersection control)
- ramp terminus (merge control)

In addition, several general controls will dictate horizontal alignment on interchange ramps. The following sections discuss all horizontal alignment criteria for ramps.

48-5.03(02) General Controls [Rev. Mar. 2025]

The following will apply to the horizontal alignment of all ramp elements:

1. Superelevation Rates (Rural). For non-loop ramps in rural areas, the superelevation rate will be based on an e_{\max} of 6% or 8% and open-road conditions. See the applicable design superelevation rate figures in Chapter 305 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 4%, 6% or 8%, depending on site conditions. For open-roadway conditions, e_{\max} of 6% or 8% should be used. See the applicable design superelevation rate figures in Chapter 305
3. Superelevation Transitions. Open-road conditions will also apply for transitioning to and from the needed superelevation on ramps. This includes the maximum relative gradients..

The methodology presented in Chapter 305 is used to calculate the superelevation runoff and tangent runout distances with the following modifications.

- a. One-Lane Ramps. When rotated about the centerline, the width of rotation (W) is 12 ft or one-half the travelway width, whichever is greater.
- b. Two-Lane Ramps. The width of rotation (W) is assumed to be one-half of the widest travelway, which is determined by the minimum radius ($R = 180\text{-ft}$) for the lowest ramp design speed ($V = 25\text{ mph}$) ($0.5 \times 27 = 13.5\text{-ft}$).

A 60-ft vertical curve should be provided at superelevation diagram P.I. locations.

4. Minimum Length of Design Superelevation. The designer should not superelevate curves on ramps such 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 about 120 ft. This distance corresponds to the 60-ft vertical curve provided at the superelevation diagram P.I. locations.
5. Axis of Rotation. This will typically be about the centerline of the ramp travelway for two-lane ramps and may be about either the centerline or the outside edge of travelway for single-lane ramps.
6. Shoulder Superelevation. The criteria presented in Chapter 305 for superelevating high-side shoulders and low-side shoulders will not apply to superelevated curves on ramps. When superelevated, the entire ramp width, including shoulders, should have the same cross slope. See Figures 48-5B, Single Lane Ramp Typical Section and 48-5C Multi-Lane Ramp Typical Section.
7. Reverse Curves. To meet restrictive right-of-way requirements, ramps may be designed with reverse curves. Desirably, these reverse curves should be designed with a normal tangent section between. 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 be the sum of the respective superelevation runoff lengths located on the tangent. It is not acceptable for the PT and PC to be coincident as this would not allow for an adequate transition of superelevation between the curves. See Chapter 305 for more information on superelevation at reverse curves.
8. Sight Distance. Section 305-5.0 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. There should be a clear view of the entire exit terminal, including the exit nose and a section of the ramp roadway beyond the gore.

48-5.03(03) Freeway/Ramp Junctions

Horizontal alignment at freeway/ramp junctions is based on open-road conditions. This is discussed in Section 48-4.0.

48-5.03(04) Ramp Proper (Directional Ramps)

Directional ramps refer to those ramps which 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.

The ramp proper, for the purpose of horizontal alignment, is considered to begin at the gore nose on exit ramps and to end approximately 200 ft before the dimension nose on entrance ramps. See the discussion in Section 48-5.03(01) to determine whether open-road conditions or turning roadway conditions apply to the horizontal alignment on directional ramps.

48-5.03(05) Ramp Proper (Loop Ramps)

Loop ramps are those ramps on the interior portions of cloverleaf and partial cloverleaf interchanges. The ramp proper is considered to begin at approximately the physical nose on exit ramps and to end at approximately the physical nose on entrance ramps. Because of the normally restrictive conditions for loop ramps, the curve radii are typically less than 300-ft. Therefore, it is desirable to use open-road conditions for horizontal alignment; although, typically, it is more practical to use turning roadway conditions. See the discussion on transition curves in Section 48-2.02(02) and 48-5.03(02).

48-5.03(06) Ramp Terminus (Intersection Control)

Interchange ramps may end at at-grade intersections. These may be stop control or signal control. If horizontal curves on the ramps are relatively close to the intersection, a design speed for the curve should be selected which is appropriate for expected operations at the curve. For these curves, the radius will determine whether open-road or turning roadway conditions apply. For $R \geq 300$ -ft, use open-road conditions. For $R < 300$ -ft, open-road conditions are desirable; turning roadway conditions are acceptable.

48-5.03(07) 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-road conditions. Profiles of highway ramp terminals should desirably be designed with a platform on the ramp side of the approach nose or merging end. This platform should be at least 200 ft in length. It should have a profile that does not greatly differ from that of the adjacent traffic lane.

48-5.04 Vertical Alignment [Rev. Jan. 2020]

48-5.04(01) Grades [Rev. Jan. 2020]

Maximum grades for vertical alignment on ramps cannot be as definitively expressed as those for highway mainline. General values of limiting gradients are 3% to 5% but, for any one ramp, the selected gradient is dependent upon a number of factors. These include the following:

1. The flatter the gradient on the ramp, the longer it will be. At restricted sites (e.g., loops), it may be necessary to provide a steeper grade to shorten the length of ramp.
2. The steepest gradients should be designed for the center portion of the ramp. Freeway/ramp junctions and landing areas at at-grade intersections should be as flat as practical.
3. Short upgrades of as much as 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. They may, however, safely exceed these values by 1%, with 6% considered to be a maximum. The 6% downgrade should only be used in extreme conditions and where restrictive geometric elements are clearly visible to the driver.
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. However, adequate sight distance is more important than grade control.
6. For high-speed system interchanges ramps, the maximum grade for both upgrades and downgrades should be limited to 4 percent. Grades steeper than 4 percent require concurrence from the Corridor Development Office.

48-5.04(02) Vertical Curvature

Vertical curves on ramps should be designed the same as 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 the lower end and with a crest vertical curve at the upper end. 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, the length of curve should be determined using a design speed intermediate between those on the ramp and the highway. See Chapter 44 for details on the design of vertical curves.

48-5.05 Roadside Safety

The criteria in Chapter 49 (e.g., clear zones, barrier warrants) will apply to the roadside safety design of interchange ramps. One special situation is the requirement for a median barrier between adjacent on/off ramps (e.g., between the outside directional ramp and inside loop ramp for a cloverleaf interchange). This will be determined on a case-by-case basis. This situation typically occurs at full or partial cloverleaf interchanges.

48-5.06 Ramp Location on a Curve

Freeway entrances and exits should be located on tangent sections wherever possible in order to provide maximum sight distance and optimum traffic operation. Where curve locations are necessary, only parallel entrance and exit ramps should be used. Ramp entrance tapers should be the same length as if on a tangent section with the width of the taper pro-rated along the length of the taper. The minimum gore nose width of 20-ft should be maintained and the degree of divergence may have to be adjusted depending on the curvature. If the designer has to modify a ramp configuration, documentation of the geometrics to accommodate the required acceleration/deceleration lengths and sight distance should be provided to the satisfaction of INDOT.

48-6.0 OTHER INTERCHANGE DESIGN CONSIDERATIONS

48-6.01 General

The designer should consider the following.

1. Design Year. The design year for the minor road intersecting the freeway should be the same as used for the freeway. The termination of other roads and streets in the area may generate a significant increase in traffic on the crossing facility.
2. Over versus Under. The decision on whether the freeway should go over or under the crossroad is normally dictated by topography. If the topography does not favor one over the other, the following factors can be used as a guide to determine which highway should cross over the other.
 - a. The designer should consider which alternative will be more cost effective to construct. Some elements are the amount of fill, grading, span lengths, angle of skew, gradients, sight distances, geometrics, constructability, traffic control and costs.
 - b. One benefit of the crossroad going over the freeway is that this may improve the ramp gradients. As drivers exit the freeway, they will normally tend to slow down going up an exit ramp and speed up going down an entrance ramp.
 - c. The alternative which provides the highest design level for the major road should be selected. Typically, the crossing road has a lower design speed; therefore, the minor road typically can be designed with steeper gradients, lesser widths, reduced vertical clearance requirements, etc.
 - d. If any crossings and/or structures are planned for a future date, the mainline should go 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.
3. 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.
4. Grading. The designer should consider the grading around an interchange early in design. Properly graded interchanges allow the overpass structures to naturally blend into the terrain. In addition, the designer needs to ensure that the slopes are not too steep to support the bridge

and roadways and that they can support plantings which prevent erosion and enhance the appearance of the area. Flatter slopes also allow easier maintenance.

48-6.02 Freeway Lane Drops

A reduction in the basic number of lanes (lane drops) may be made beyond a principal interchange involving a major fork or at a point downstream from an interchange with another freeway. This reduction may be made provided the exit volume is sufficiently large enough to change the basic number of lanes beyond this point on the freeway route as a whole. Another situation where the basic number of lanes may be reduced is where a series of exits, as in outlying areas of a city, causes the traffic load on the freeway to drop sufficiently to justify the lesser number of lanes. Dropping a basic lane or an auxiliary lane may be accomplished at a two-lane exit ramp or between interchanges. Lane reductions should not be made between and within interchanges simply to accommodate variations in traffic volumes. Instead, auxiliary lanes, as needed, are added or removed from the basic number of lanes.

Figure [48-6A](#) illustrates the recommended design of a lane drop beyond an interchange. The following criteria are important when designing a freeway lane drop.

1. Location. The lane drop should occur approximately 2000 ft – 3000 ft beyond the end of entry-ramp taper of the previous interchange. Under restricted conditions, the MUTCD signing distance is acceptable. This distance allows 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, a lane should not be dropped on a horizontal curve or where other signing is required, such as for 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 48-3.0, a lane should not be dropped at an exit unless there is a large decrease in traffic demand for a significant length of freeway.

2. Transition. The desirable transition taper rate is 70:1. The minimum acceptable taper rate being 50:1 (see Figure [48-6A](#)).
3. Sight Distance. Decision sight distance (DSD) should be available to any point within the entire lane transition. See Section 42-2.0 for applicable DSD values. When determining the availability of DSD, the desirable height of object will be 0.0 in. (the roadway surface); it is acceptable to use 6-in. This criterion would favor, for example, placing a freeway lane drop within a sag vertical curve rather than just beyond a crest.

4. Right-Side versus Left-Side Drop. Right-side freeway lane drops are preferred in urban settings; however, a left-side lane reduction may be more practical in rural settings where truck traffic is usually predominant in the right lanes and overall traffic volumes are less in the left lanes.
5. Shoulders. The full-width right shoulder will be maintained through a right-side lane drop. If a left-side lane drop will be used to reduce the number of lanes from three to two, the left shoulder will be reduced from 10 ft (or 12 ft) to 4 ft. The full 10-ft left shoulder should be maintained for a distance of approximately 300 ft beyond the merge point of the dropped lane. The additional distance provides an area to allow a driver, who may have missed the signing, an opportunity to safely merge with the through traffic. The shoulder should then be transitioned from 10 ft to 4 ft over a minimum length of 100 ft. The additional length of shoulder beyond the merge point should be full depth pavement. (See Figure [48-6A](#)).

48-6.03 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:

1. remove weaving maneuvers from the mainline and transfer them to the slower 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 when traffic volumes are so high that the interchange without them cannot operate at an acceptable LOS, 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 [48-2I](#) 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 45 to 50 mph; however, it should desirably be within 10 mph 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 barrier between the two (e.g., 20 to 25 ft).

48-6.04 Frontage Roads

The designer must consider the impact of frontage roads, where present, on interchange design. At some urban interchanges, it may be impractical to separate the intersections of the ramp and frontage road with the crossing road. In these cases, the only alternative is to merge the ramp and frontage road before the intersection with the crossing road. This can apply to either the exit or entrance ramp.

Figure [48-6B](#) provides the basic schematic for this design. This design 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 [48-6B](#) also presents general guidelines which 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. See Transportation Research Record 682 *Distance Requirements for Frontage-Road Ramps to Cross Streets: Urban Freeway Design* for additional information.

Distance “B” in Figure [48-6B](#) 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.

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

1. **Slip Ramps.** Slip ramps may be used to connect the freeway with 1-way frontage roads before (or after) the intersection with the crossing road. Newly constructed slip ramps to a 2-way frontage road are unacceptable because they may induce wrong-way entry onto the freeway and may cause accidents at the intersection of the ramp and frontage 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 [48-6C](#), Typical Access Control for a Partial Cloverleaf Interchange, 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.

Section 45-7.0 discusses overall design criteria for frontage roads (e.g., functional class, outer separation).

48-6.05 Ramp/Crossing Road Intersection

At service interchanges, the ramp will typically end with an at-grade intersection at the cross road. In general, the intersection should be treated as described in Chapter 46. This will involve a consideration of capacity and the physical geometric design elements such as sight distance, angle of intersection, acceleration lanes, channelization and turning lanes. However, several points require special attention in the design of the ramp/crossing road intersection:

1. 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, special attention should be given to providing sufficient capacity and storage for an at-grade intersection or a merge with the crossing road. This may require adding additional lanes at the intersection or on the ramp proper, or it could involve traffic signalization where the ramp traffic will be given priority. The analysis must also consider the operational impacts of the traffic characteristics in either direction on the intersecting road.
2. Sight Distance. Section 46-10.0 discusses the criteria for intersection sight distance. These criteria also apply to a ramp/crossing road intersection. Special attention must be given to the location of the bridge pier, abutment, sidewalk, bridge rail, roadside barrier, etc. These may present major sight distance obstacles. The bridge obstruction and the required intersection sight distance may result in the need to relocate the ramp/crossing road intersection.
3. Wrong-Way Movements. Wrong-way movements may originate at the ramp/crossing road intersection. The intersection must be properly signed and designed to minimize the potential for a wrong-way movement (e.g., channelization).

4. Turn Lanes. Additional turn lanes are often required at the end of ramp. Section 46-4.0 provides information on the design of turn lanes at intersections at-grade.
5. Distance between Free-Flow Terminal and Structure. The terminal of a ramp should not be near the grade-separation structure. If it is not practical to place the exit terminal in advance of the structure, the existing terminal on the far side of the structure should be well-removed. When leaving, drivers should be permitted some distance after passing the structure in which to see the turnout and begin the turnoff maneuver. Decision sight distance is recommended where practical. The distance between the structure and the approach nose at the ramp terminal should be sufficient for exiting drivers to leave the through lanes without undue hindrance to through traffic.

48-6.06 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.

Crossroads with interchange access typically provide the catalyst for development and traffic generators. To maintain the integrity of the freeway terminals it has become more apparent to extend access control beyond the ramp terminal nominal distance of 100-ft to 500-ft. Studies have shown that annual crash rates on cross roads with the first access located within 300 ft from the off ramp are at least 50% higher than those where the first access is located at the 300-750-ft range. At a minimum, the first access point should be located in urban areas 600-ft and 750-ft in rural locations from the off ramp away from the interchange along the cross road. On multi-lane crossroad terminals, access control should protect the distance to accommodate acceleration, weaving deceleration, transitioning, and storage to the first access point from the ramp terminal (See [Figure 48-6D](#)). In fully developed urban areas, these distances may not be achievable, but efforts should be made to avoid the minimum.

Figures [48-6E](#) and [48-6F](#) show access spacing based on one or more of four types of access connections upstream and downstream of an interchange terminus with both a two-lane crossroad and multi-lane crossroad configuration. Access connection types include:

1. Nearest access (all types)
2. Right-in/right-out
3. Un-signalized, full access
4. Signalized, full access

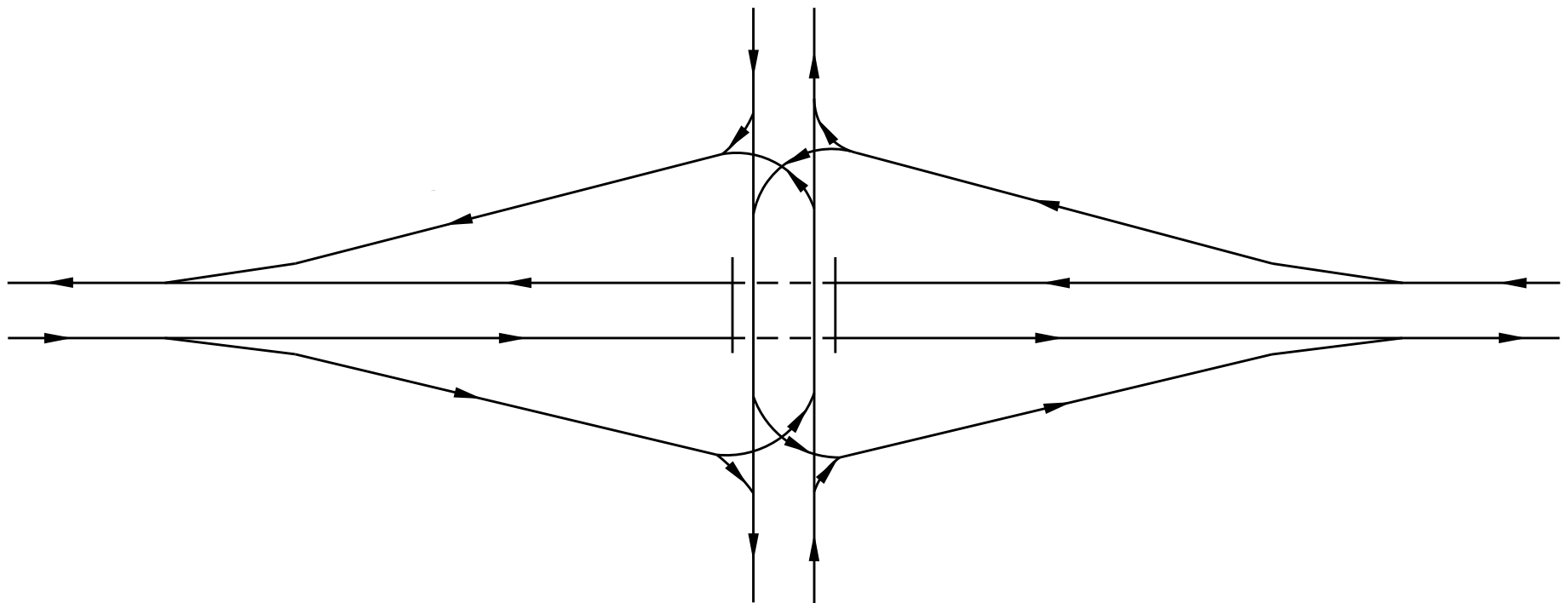
In addition, many areas have changed over the years from rural to urban. As indicated, INDOT has adopted different criteria for the access control at urban and rural interchanges. However, a change in area character alone is not a sufficient justification to alter the location of the full-access control line when an existing interchange will be rehabilitated or when INDOT receives requests for additional access points from outside interests.

Figure [48-6G](#) show the extent of limited access right of way at ramp terminals. The figure states 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 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.

Type of Intersecting Facility	Rural		Suburban	Urban
Collectors and Arterials	Service Interchanges			
Freeways	System Interchanges			

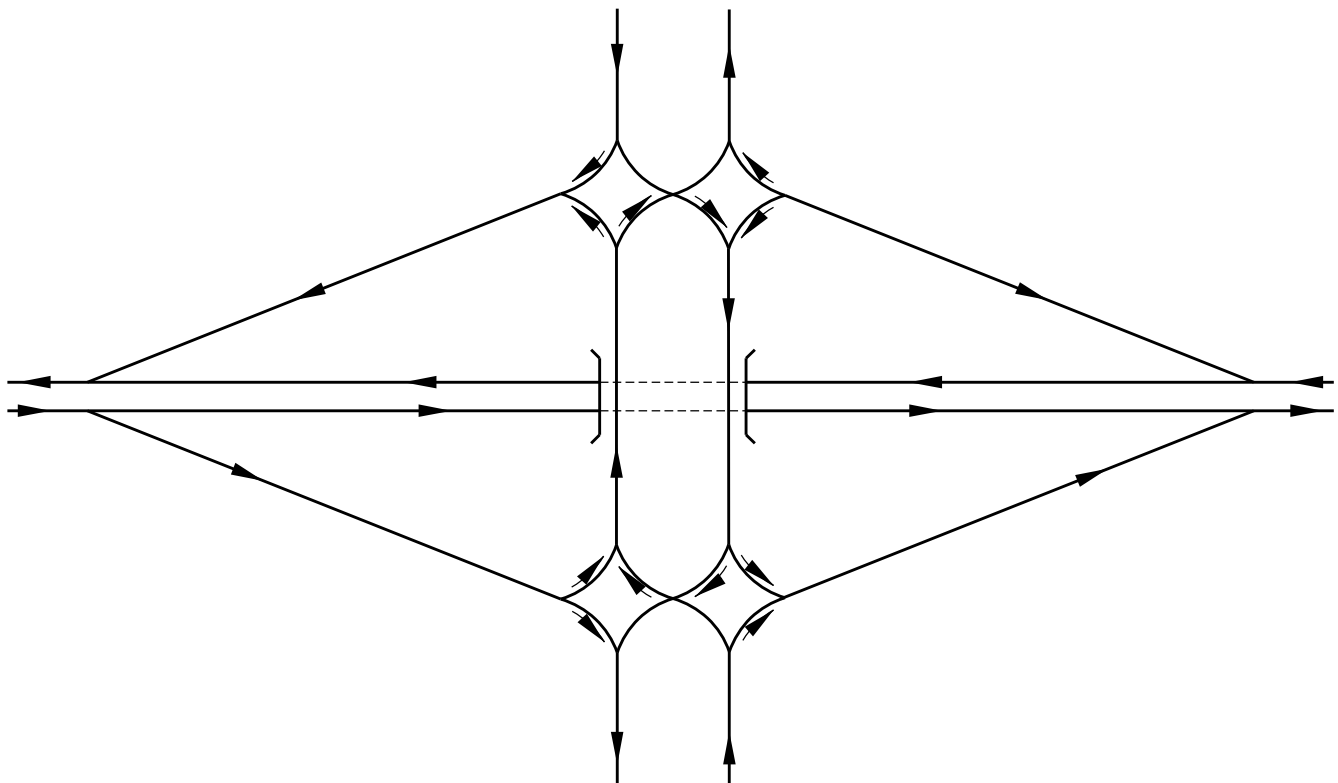
FREEWAY INTERCHANGES (BASED ON FUNCTIONAL CLASSIFICATION OF INTERSECTING FACILITY)

Figure 48-2A



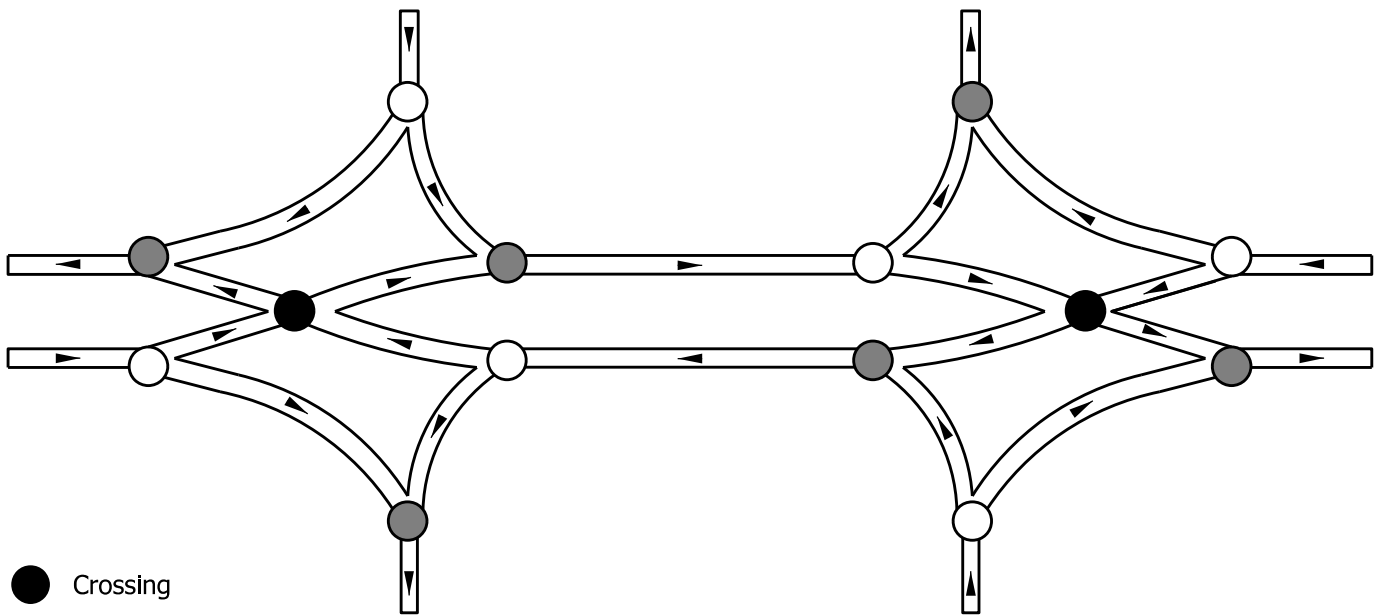
DIAMOND INTERCHANGE

Figure 48-2B



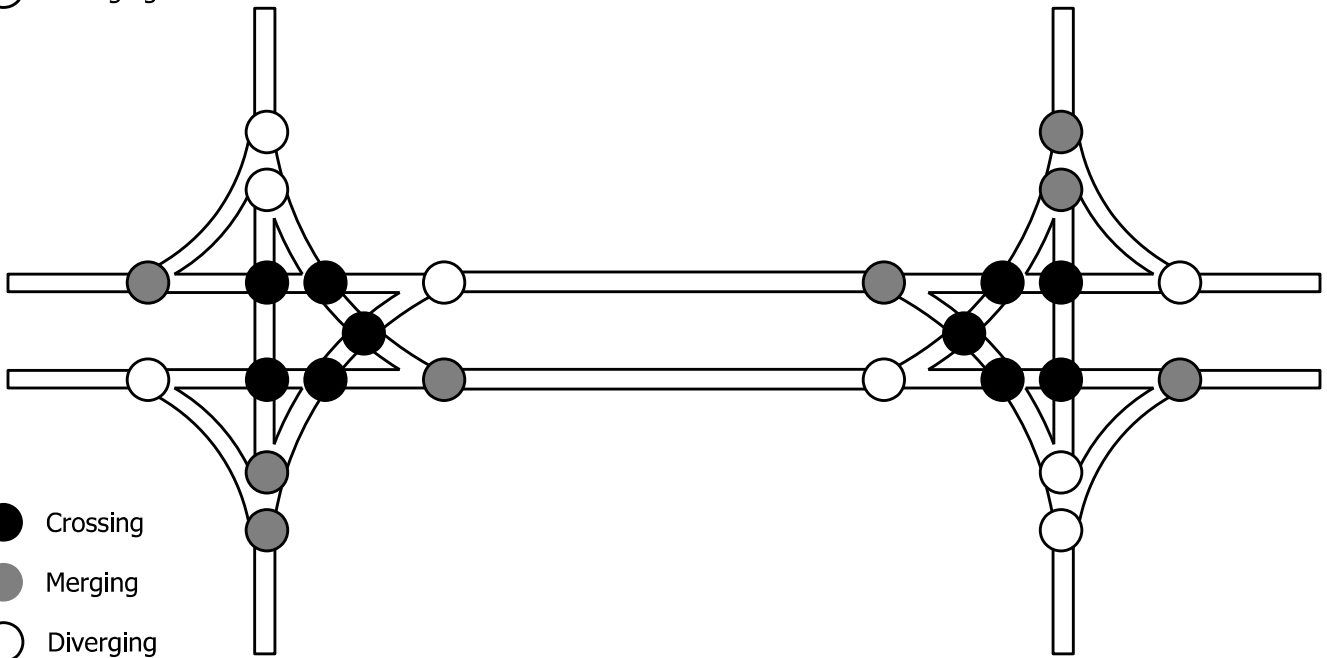
DIVERGING DIAMOND INTERCHANGE

Figure 48-2C



- Crossing
- Merging
- Diverging

Conflict Diagram for Diverging Diamond



- Crossing
- Merging
- Diverging

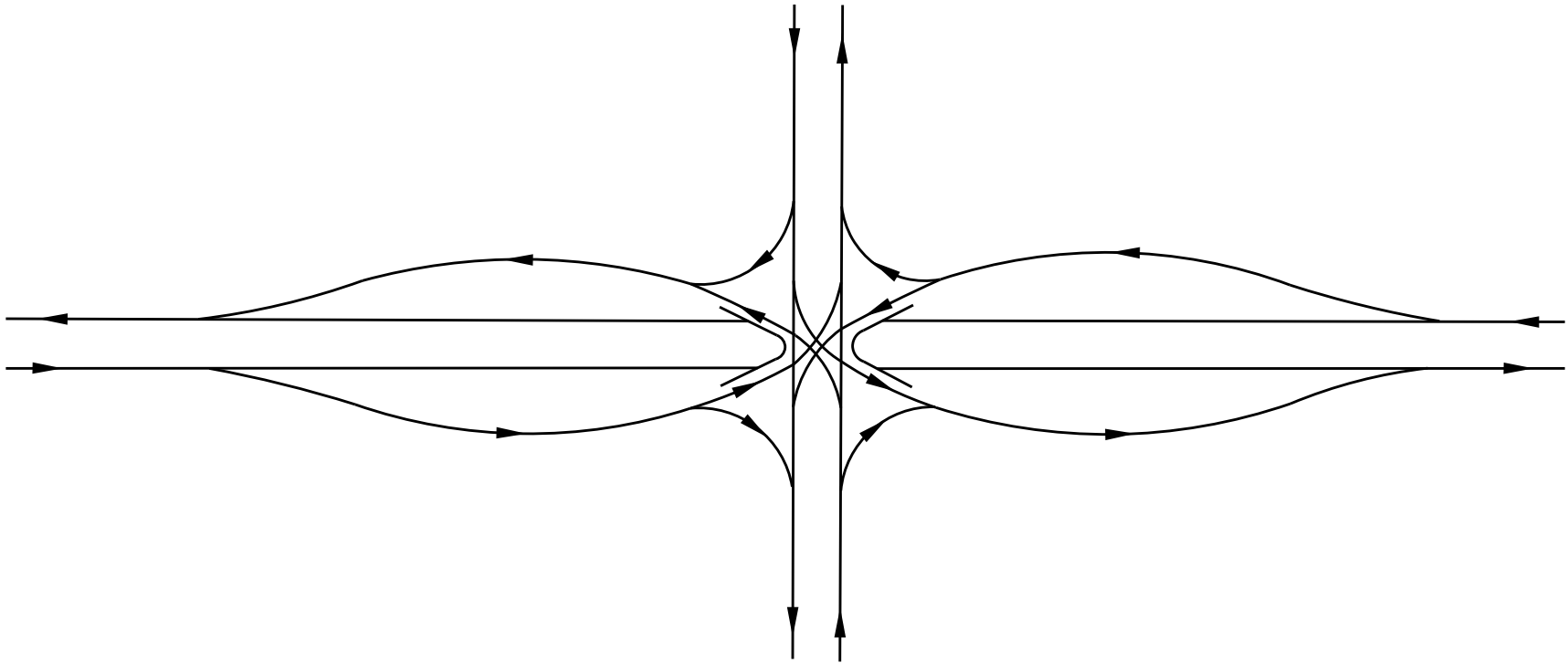
Conflict Diagram for Conventional Diamond Interchange

CONFLICT POINT COMPARISON

	Crossing	Merging	Diverging	Total
Conventional Diamond	10	8	8	26
Diverging Diamond	2	6	6	14

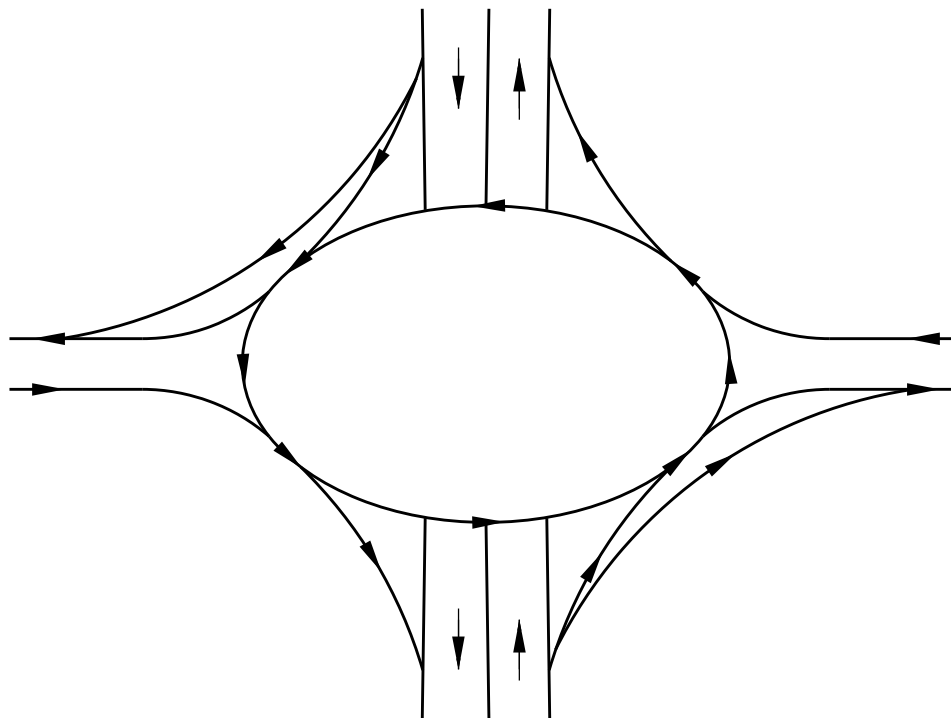
CONFLICT DIAGRAMS

Figure 48-2D



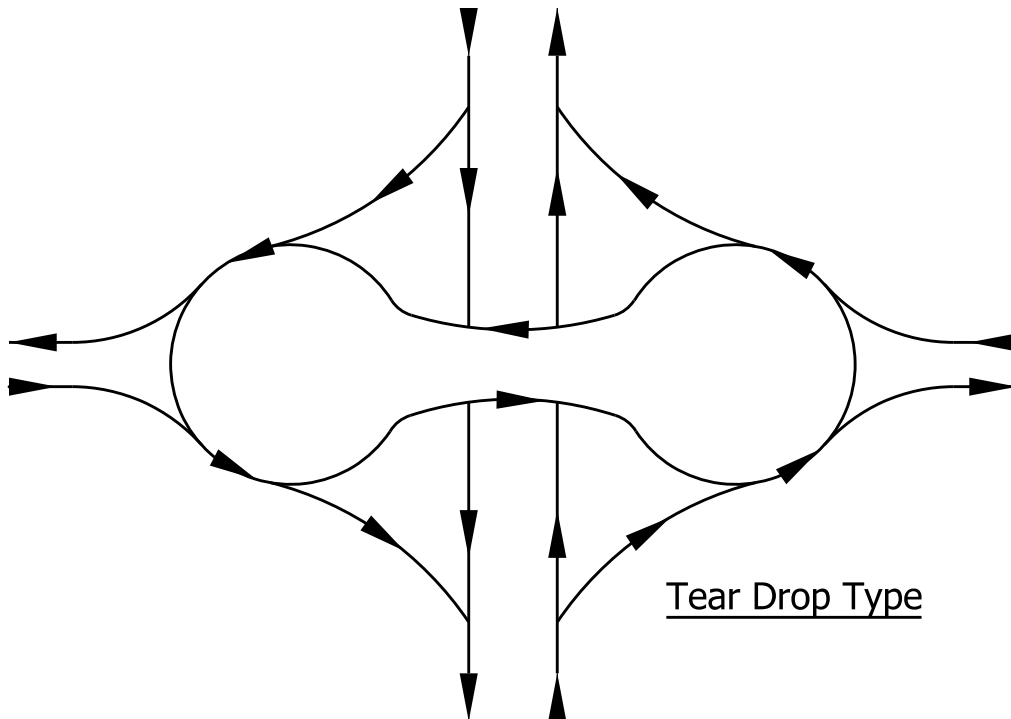
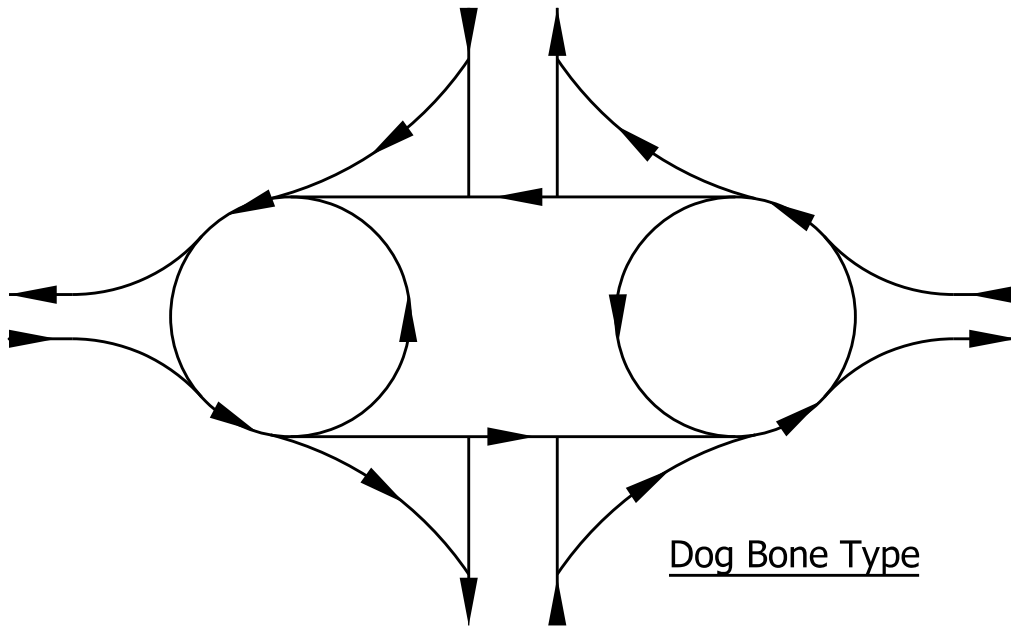
SINGLE POINT DIAMOND INTERCHANGE

Figure 48-2E



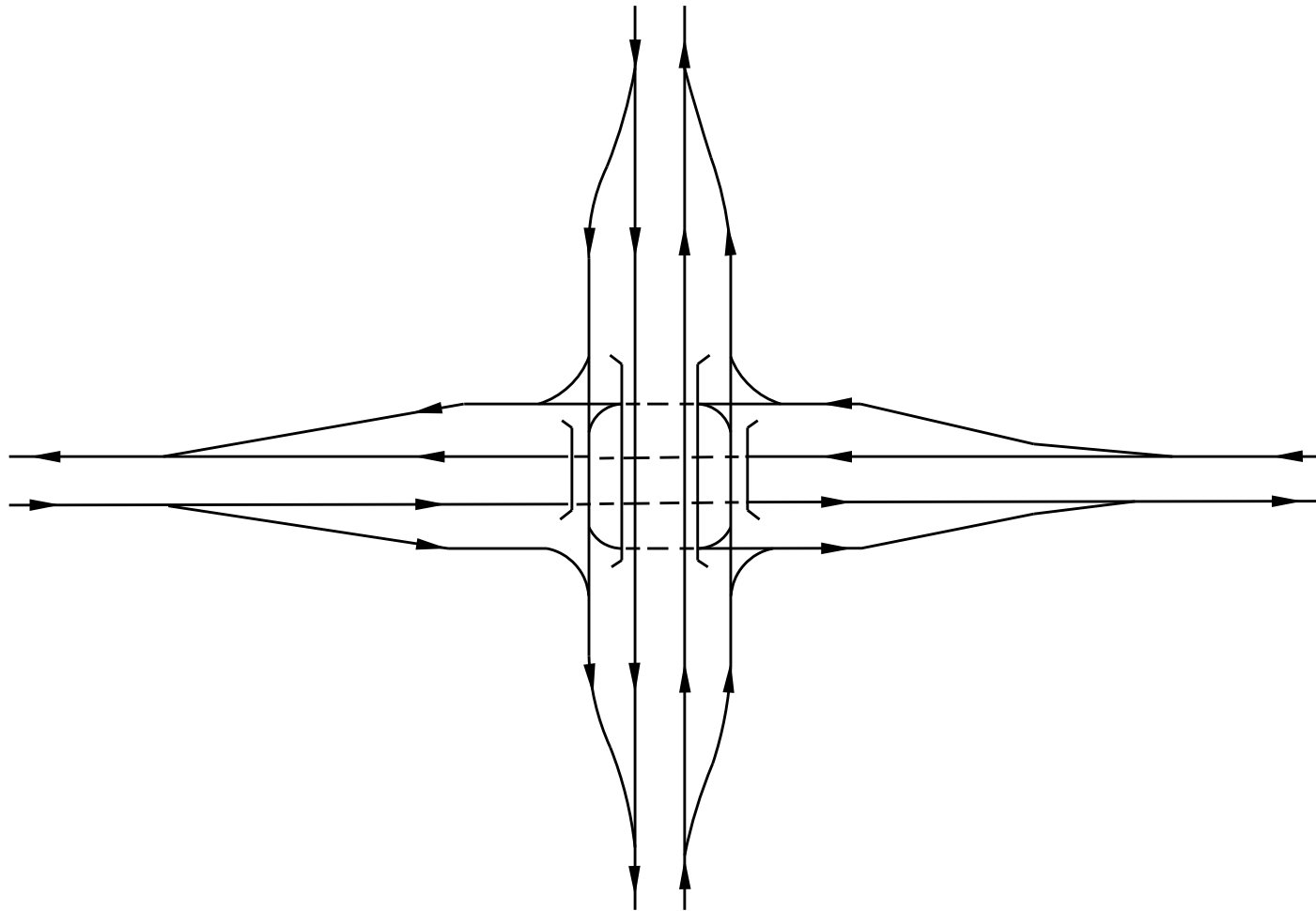
SINGLE ROUNDABOUT DIAMOND INTERCHANGE

Figure 48-2F



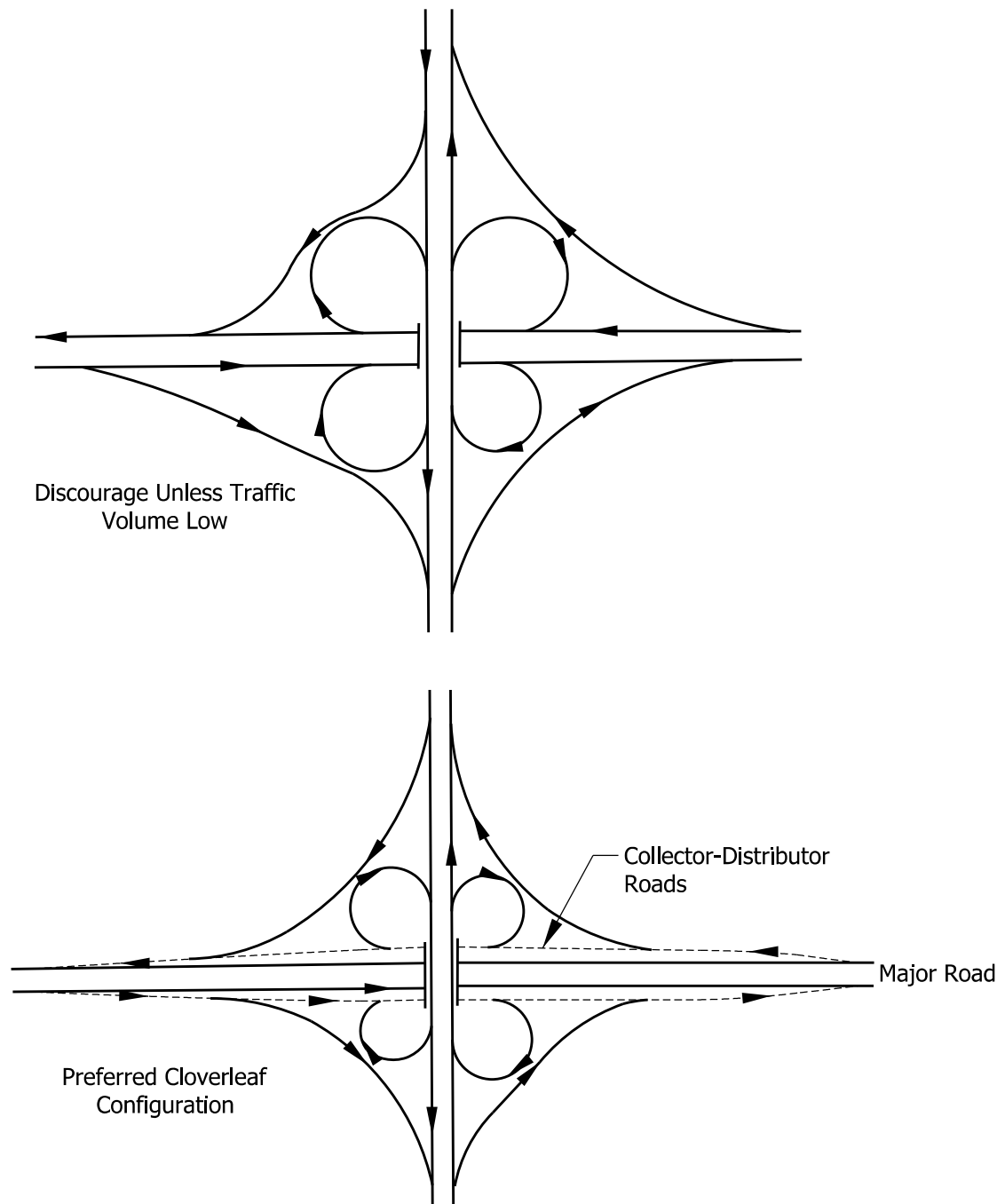
DOUBLE ROUNDABOUT DIAMOND INTERCHANGE

Figure 48-2G



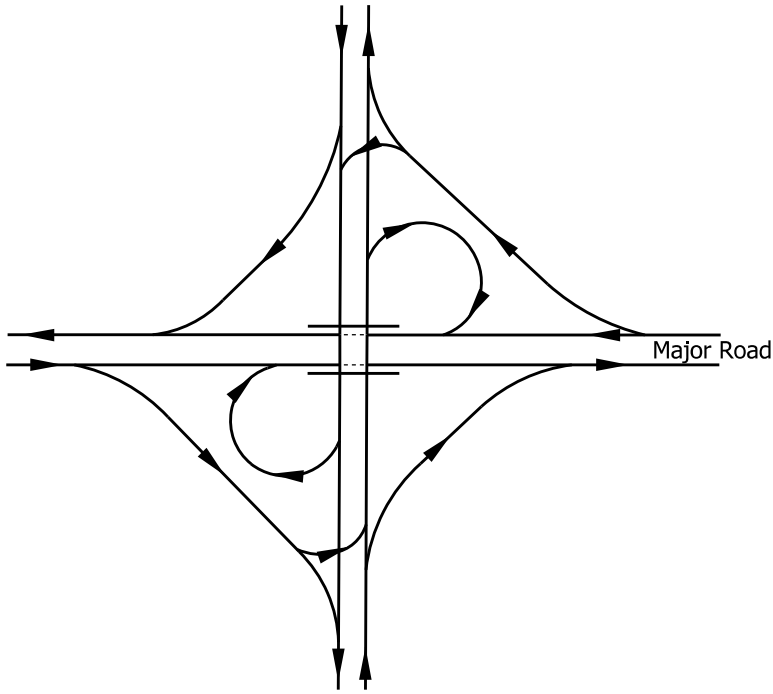
THREE-LEVEL DIAMOND INTERCHANGE

Figure 48-2H

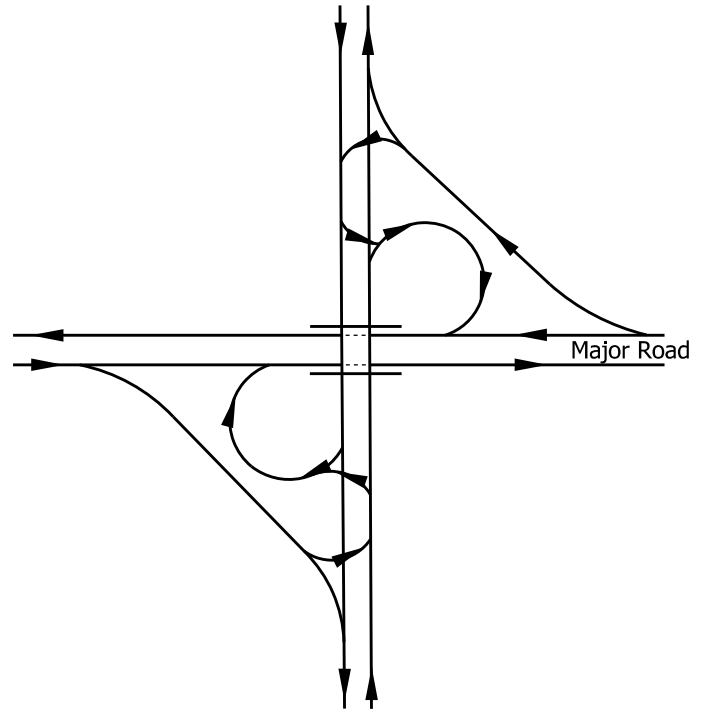


FULL CLOVERLEAF INTERCHANGE

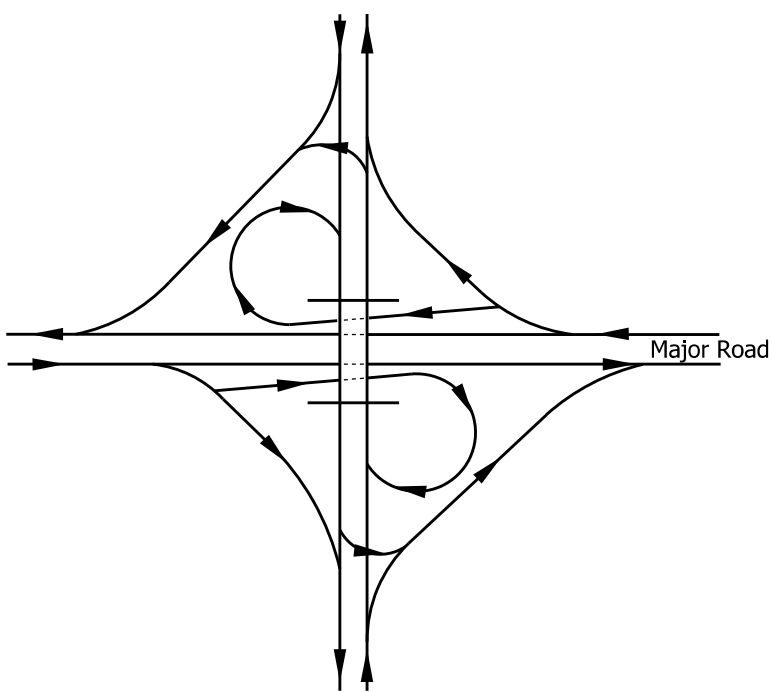
Figure 48-2I



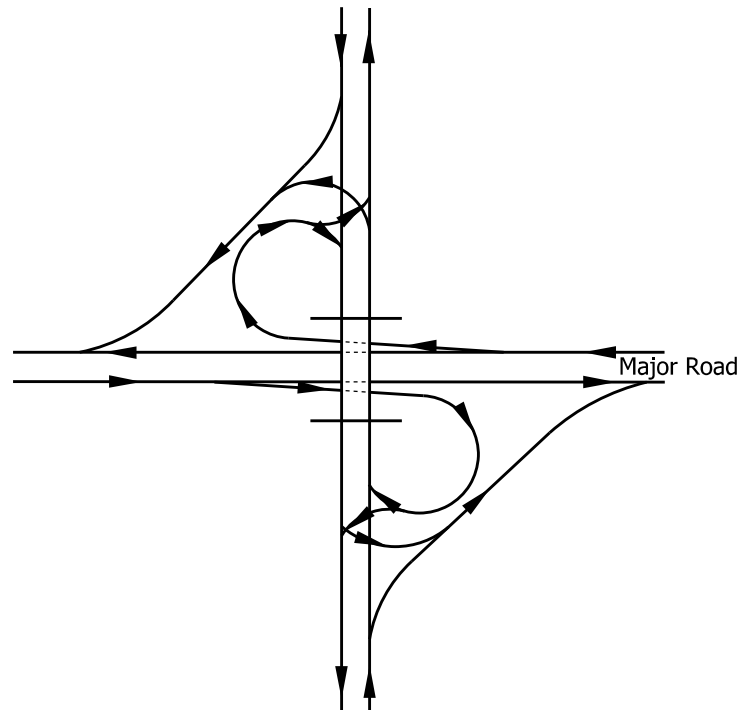
(A) ParClo - A



(B) ParClo - A (Two Quad)

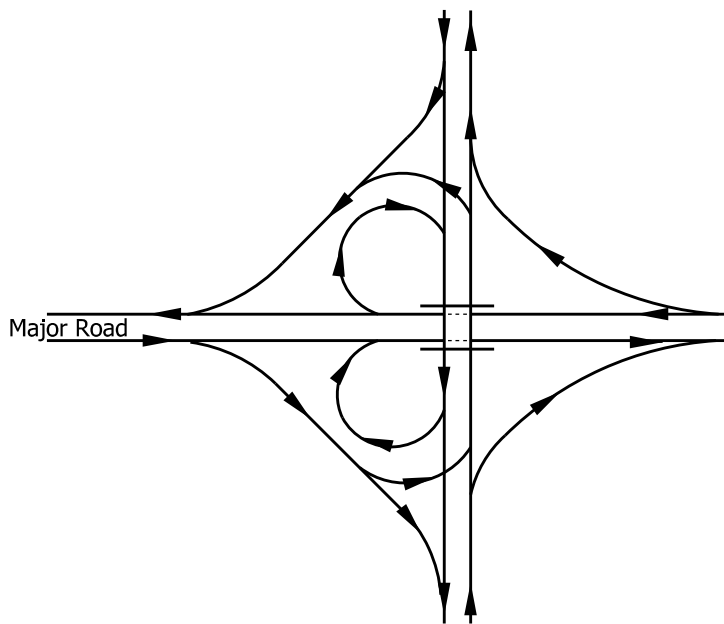


(C) ParClo - B

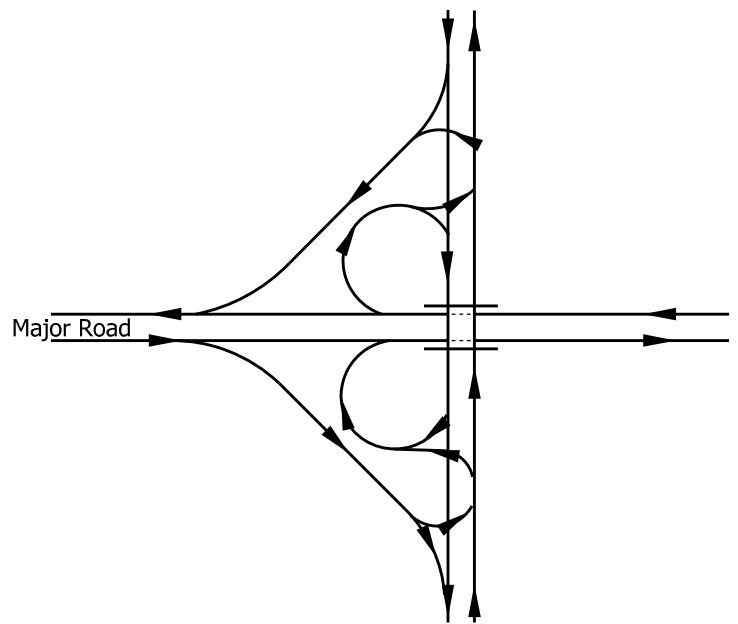


(D) ParClo - B (Two Quad)

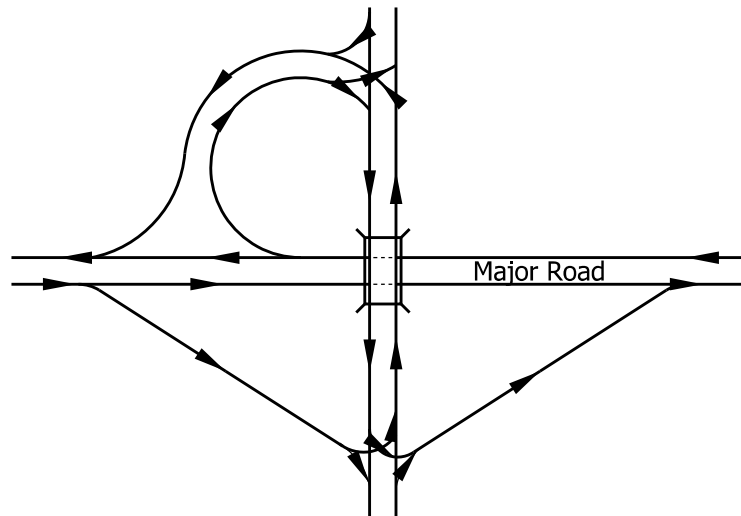
PARTIAL CLOVERLEAF (PARCLO)



ParClo - AB

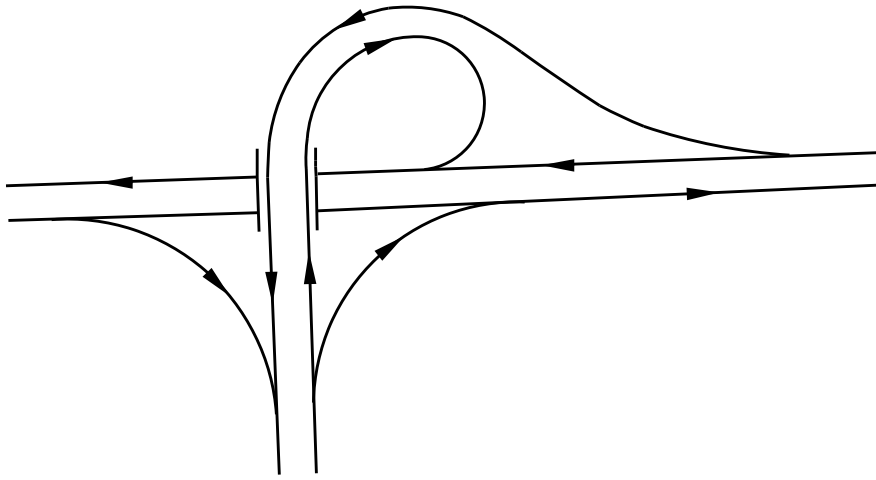


ParClo - AB (Two Quad)

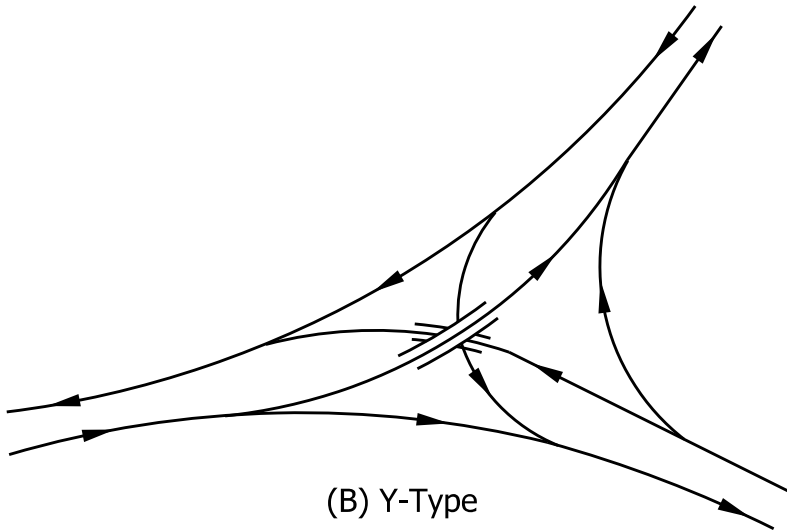


(E) Single Loop ParClo Interchange

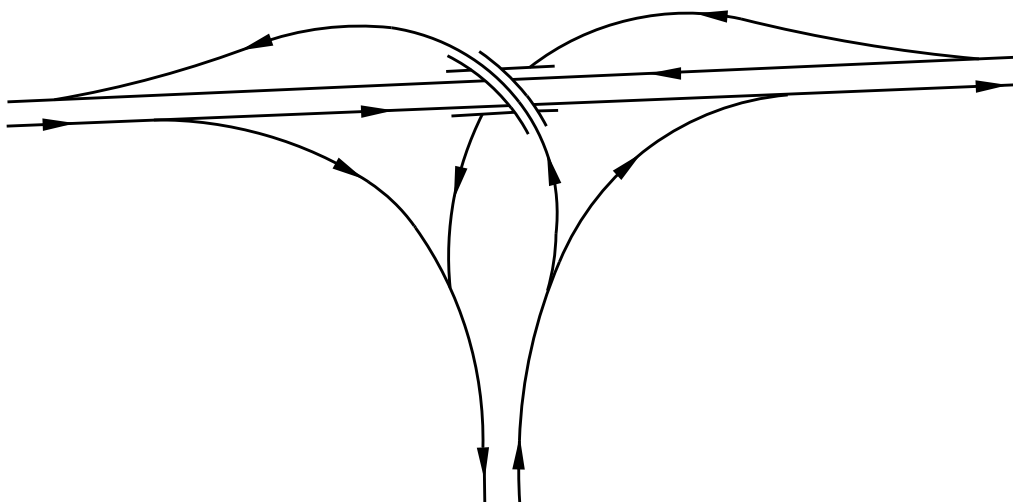
PARTIAL CLOVERLEAF (PARCLO)



(A) Trumpet or Jug Handle



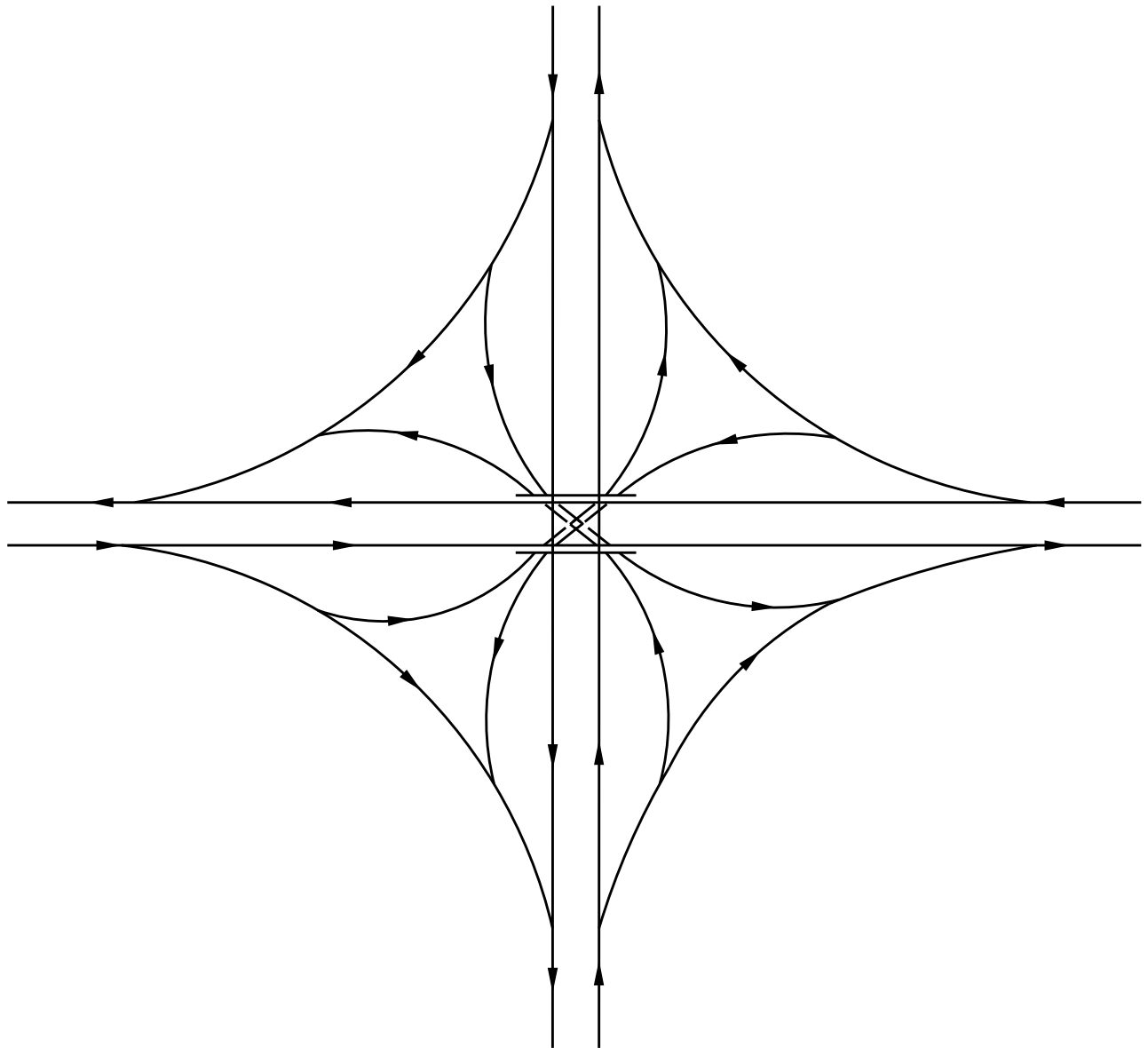
(B) Y-Type



(C) T-Type

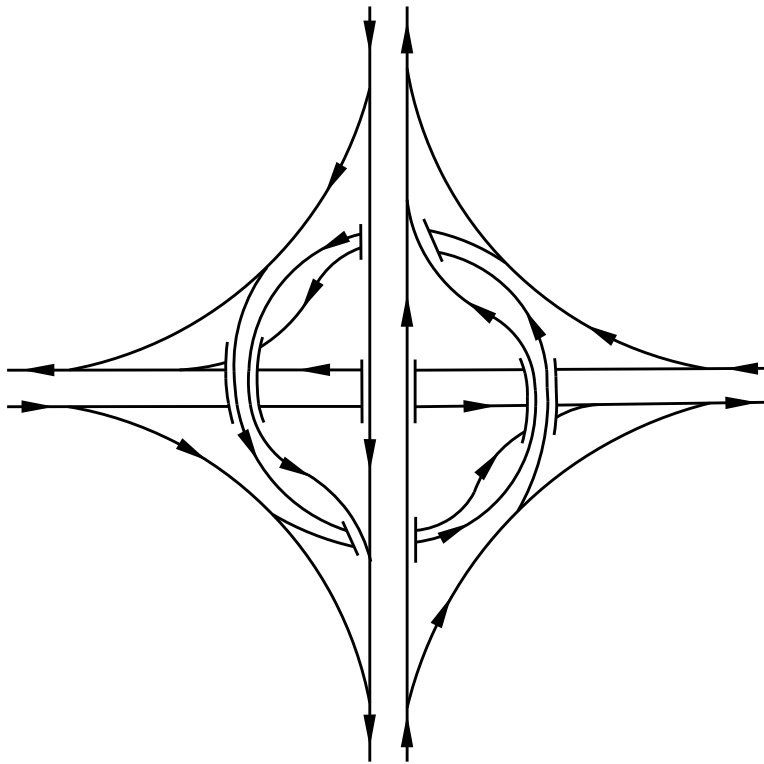
THREE-LEG INTERCHANGE

Figure 48-2K

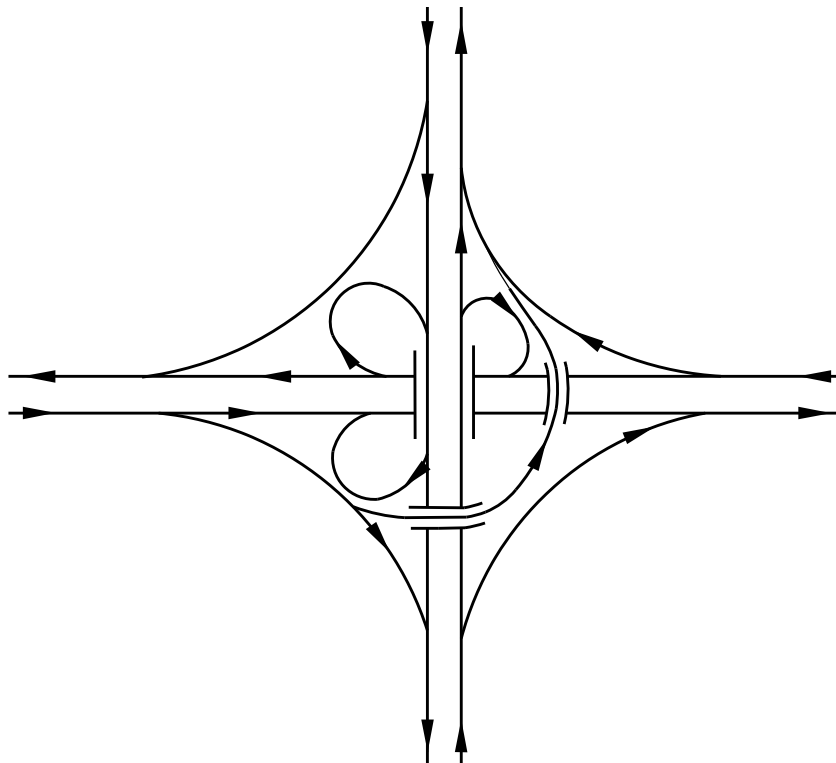


FULLY DIRECTIONAL INTERCHANGE

Figure 48-2L



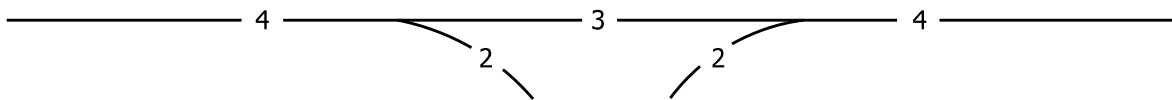
(A) Semi-direct connection
with multi-level structures



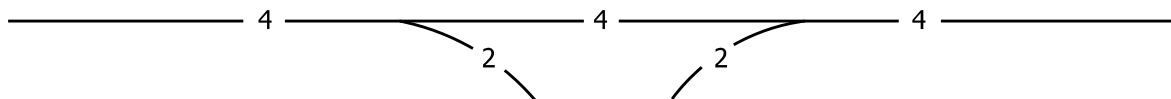
(B) Cloverleaf with Semi-direct connection

SEMI-DIRECTIONAL INTERCHANGE

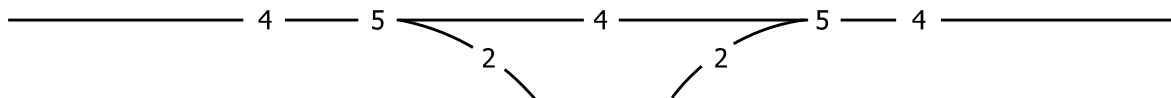
Figure 48-2M



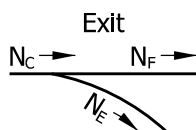
(A) Lane Balance No Compliance with Basic Number of Lanes



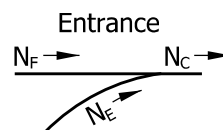
(B) No Lane Balance Compliance with Basic Number of Lanes



(C) Lane Balance and Compliance with Basic Number of Lanes



$$N_C = N_F + N_E - 1$$



$$\begin{aligned} \text{Max: } N_C &= N_F + N_E \\ \text{Min: } N_C &= N_F + N_E - 1 \end{aligned}$$

Where:

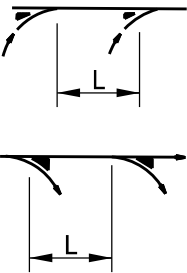
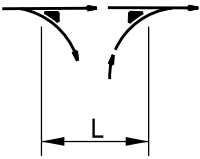
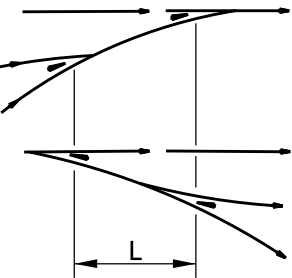
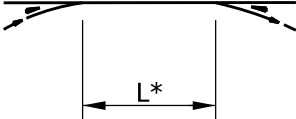
N_C = Number of Lanes for Combined Traffic

N_F = Number of Lanes on Freeway

N_E = Number of Lanes on Exit or Entrance Ramp

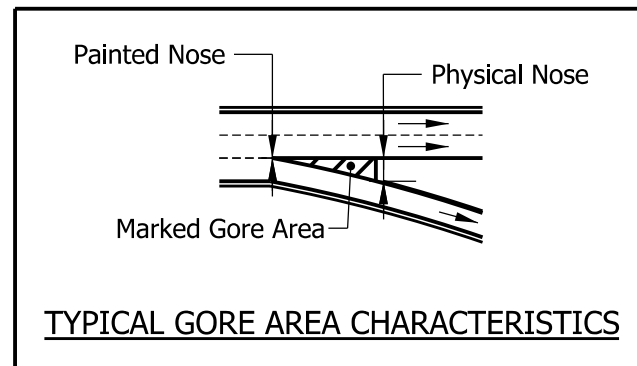
COORDINATION OF LANE BALANCE AND NUMBER OF LANES

Figure 48-3A

EN-EN or EX-EX		EX-EN		Directional Ramps		EN-EX (Weaving)			
						 *Not Applicable to Cloverleaf Loop Ramps			
Full Freeway	CDR	Full Freeway	CDR	System Interchange	Service Interchange	System to Service Interchange		Service to Service Interchange	
						Full Freeway	CDR	Full Freeway	CDR
Minimum Lengths (L) Measured Between Successive Ramp Terminals (ft)									
1000 ft	800 ft	500 ft	400 ft	800 ft	600 ft	2000 ft	1600 ft	1600 ft	1000 ft

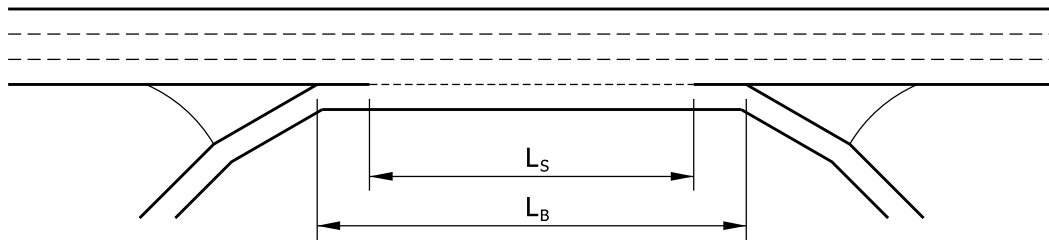
Notes: CDR - Collector Distributor Road
 EN - Entrance
 EX - Exit

The recommended minimum length (L) is based on operational experience, need for flexibility, and adequate signing. Lengths should be checked in accordance with the procedure outlined in the *Highway Capacity Manual* (HCM). Refer to the HCM for the procedure to determine the length of the weaving section. The "L" distances noted above are measured between the painted noses (theoretical gore point). For EN-EN, a minimum distance of 300 ft is recommended between the end of the taper for the first entrance ramp and the painted nose for the succeeding entrance ramp.

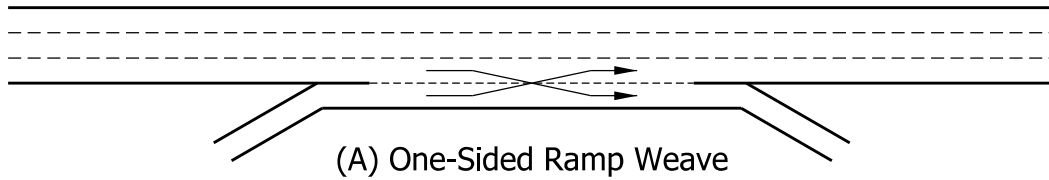


RECOMMENDED MINIMUM RAMP TERMINAL SPACING

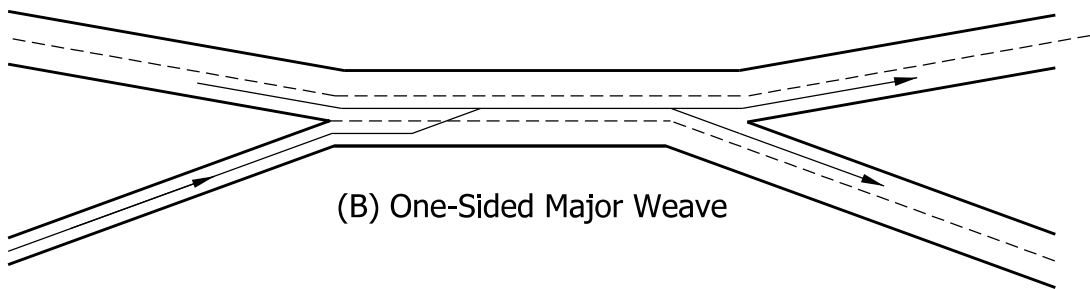
Figure 48-3B



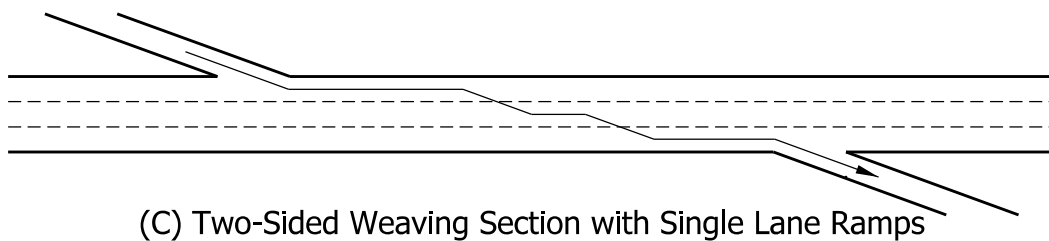
The two lengths of a weaving segment



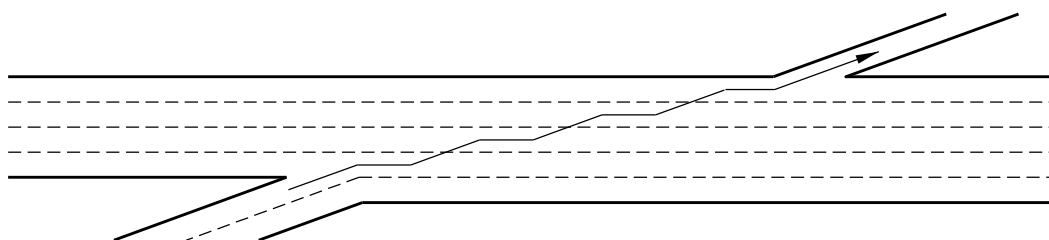
(A) One-Sided Ramp Weave



(B) One-Sided Major Weave



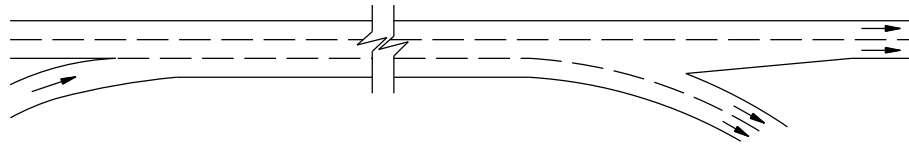
(C) Two-Sided Weaving Section with Single Lane Ramps



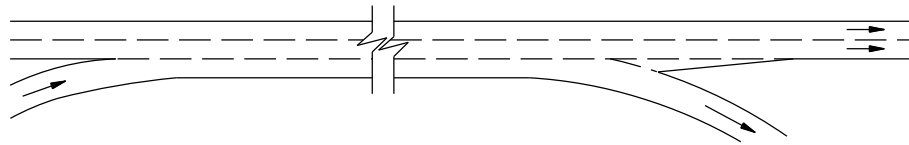
(D) Two-Sided Weaving Section with Three Lane Changes

WEAVING SEGMENTS

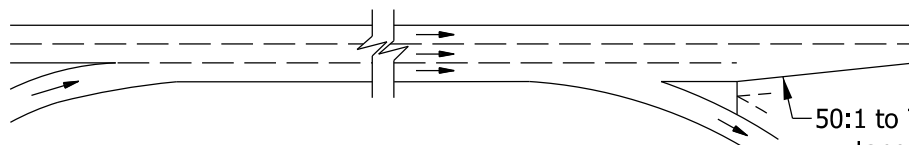
Figure 48-3C



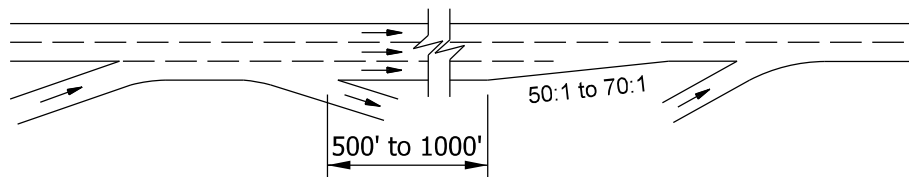
(A) Auxiliary lane dropped on exit ramp



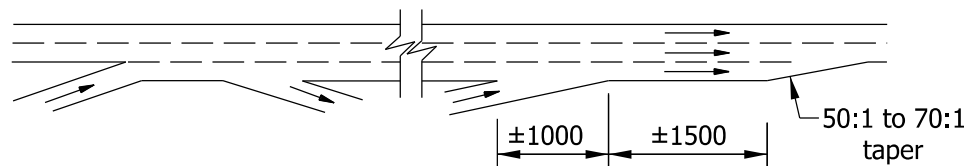
(B) Auxiliary lane between cloverleaf loops or closely spaced interchanges dropped on single exit lane



(C) Auxiliary lane dropped at physical nose



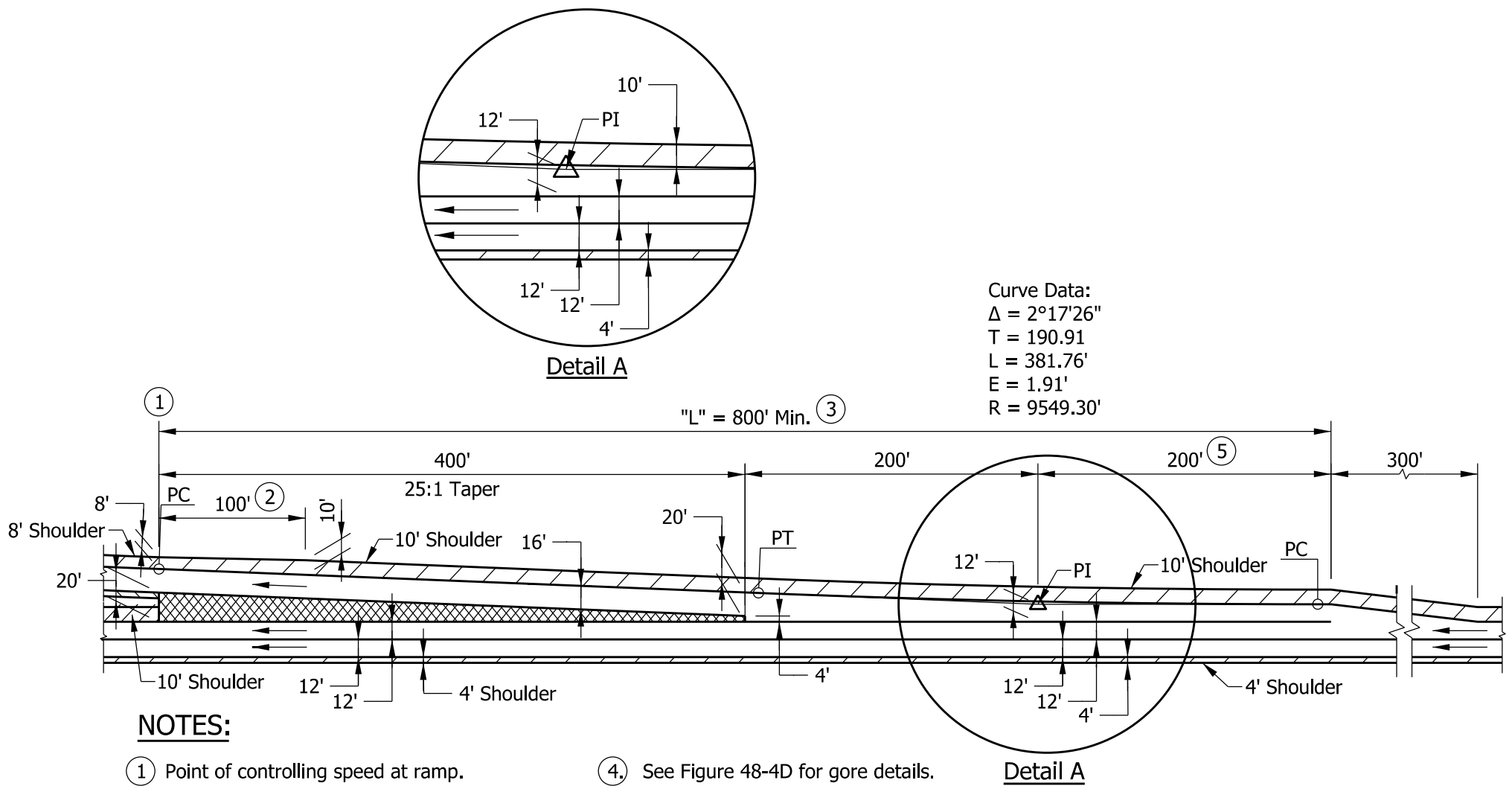
(D) Auxiliary lane dropped within an interchange



(E) Auxiliary lane dropped beyond an interchange

ALTERNATE METHODS OF DROPPING AUXILIARY LANES

Figure 48-3D



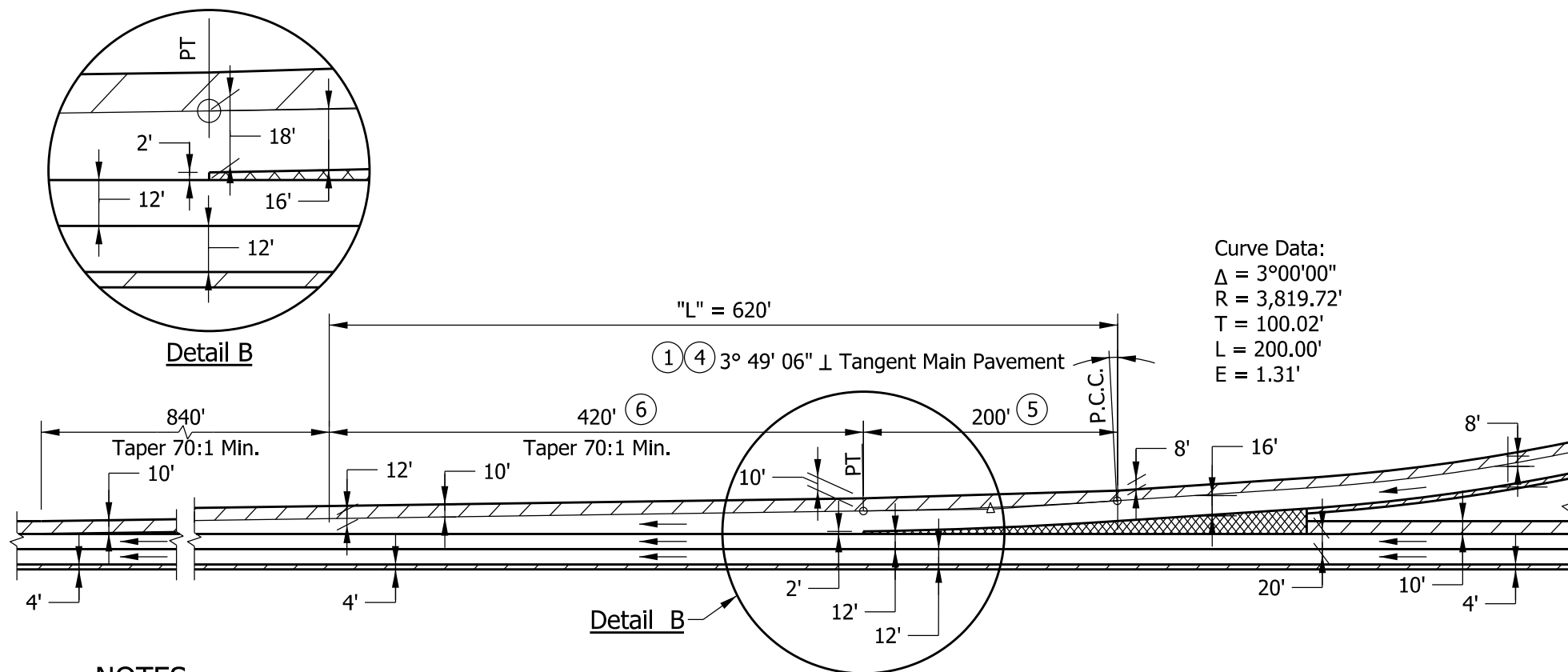
PARALLEL SINGLE LANE FREEWAY EXIT RAMP (PREFERRED) (SERVICE INTERCHANGE)

Figure 48-4A



- ## TAPERED SINGLE LANE FREEWAY EXIT RAMP (SERVICE INTERCHANGE)

Figure 48-4B

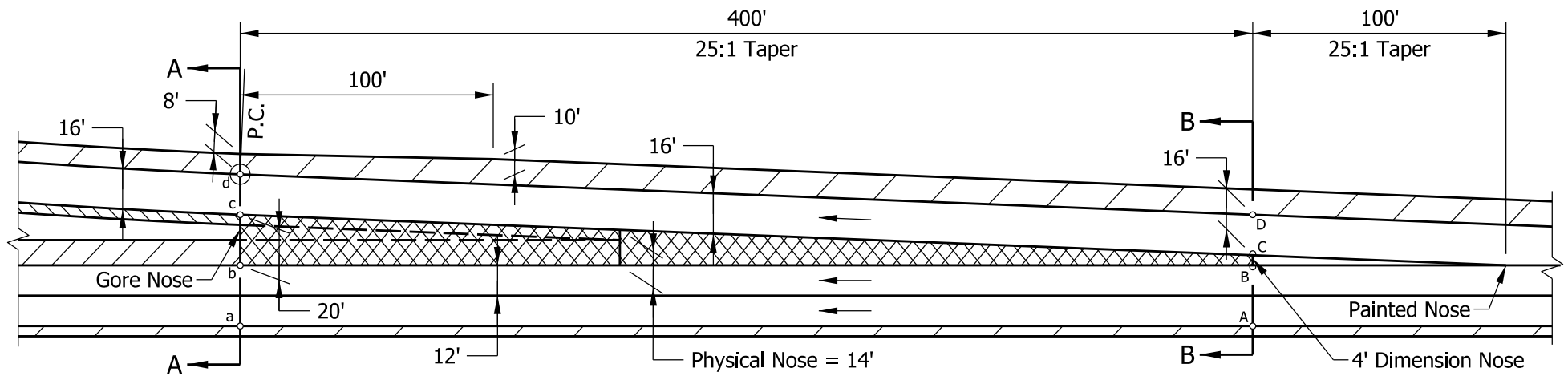


NOTES:

- ① Point of controlling speed on the ramp.
2. If the required "L" acceleration length is greater than 620 feet use a parallel single lane freeway entrance ramp (Figure 48-4C (page 1 of 2)).
3. For entrance gore details see Figure 48-4D.
- ④ For tapered design, the upstream design speed from the P.C.C. must meet the middle range design speed as shown in Figure 48-5A.
- ⑤ Transition shoulder width from 8 ft to 10 ft.
- ⑥ Transition pavement width from 18 ft to 12 ft.
7. Use of tapered single lane freeway entrance ramp configuration requires approval from the Director of Highway Design and Technical Support.
8. For ramps on curves, see Section 48-5.06.

TAPERED SINGLE LANE FREEWAY ENTRANCE RAMP (SERVICE INTERCHANGE)

Figure 48-4C (Page 2 of 2)

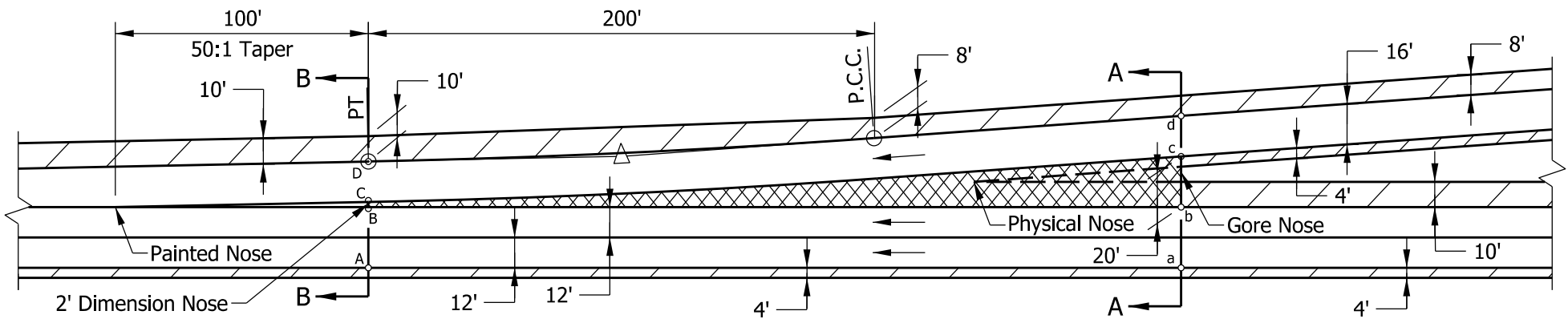


Exit Gore

Curve Data:
 $\Delta = 3^{\circ}00'00''$
 $R = 3,819.72'$
 $T = 100.02'$
 $L = 200.00'$
 $E = 1.31'$

NOTE:

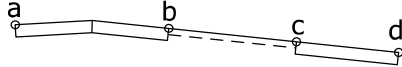
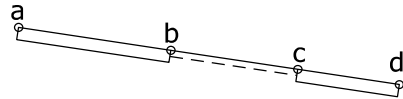
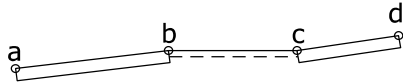
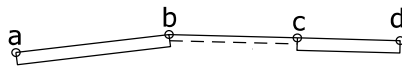
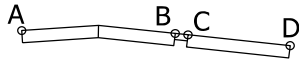
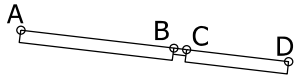
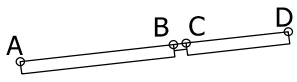
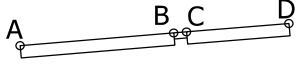
1. For ramps on curves, see Section 48-5.06.



Entrance Gore

GORE DETAILS

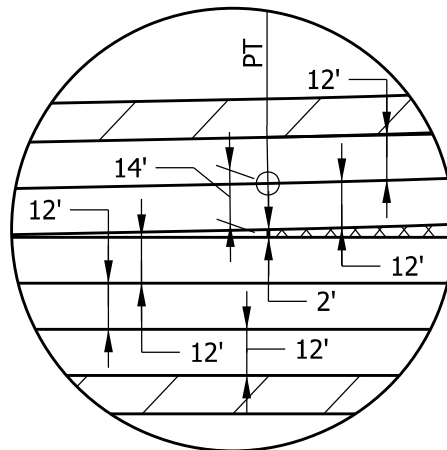
Figure 48-4D (Page 1 of 2)

When the through lanes are not superelevated	When the thru lanes are superelevated and B is lower than A	When the through lanes are superelevated and B is higher than A	
		Ramp and through lane superelevated in same direction	Ramp and through lane superelevated in opposite direction
SECTION A-A			
 <p>Points b, c & d should be progressively lower.</p>	 <p>Points b, c & d should be progressively lower.</p>	 <p>Points c should be higher than point b.</p>	 <p>Points c should be equal to or lower than point b.</p>
SECTION B-B			
 <p>Points B, C & D should be in same plane.</p>	 <p>Points A, B, C & D should be in same plane.</p>	 <p>Points A, B, C & D should be in same plane.</p>	 <p>Points A, B, C & D should be in same plane.</p>

Note: Maximum rollover between mainline pavement and gore is 5%.

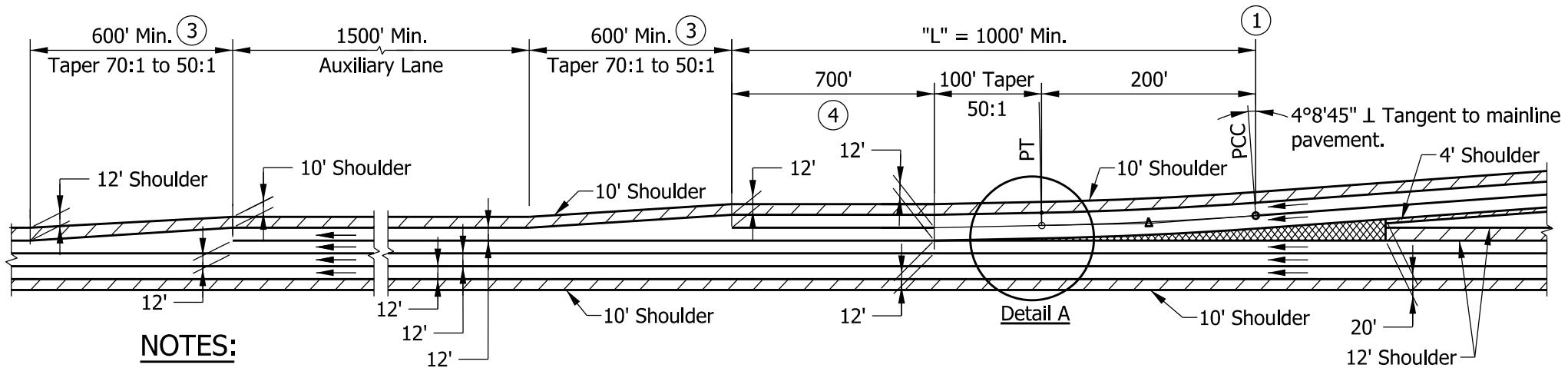
GORE DETAILS

Figure 48-4D (Page 2 of 2)



Detail A

Curve Data:
 $\Delta = 3^{\circ}00'00''$
 $R = 3,819.72'$
 $T = 100.02'$
 $L = 200.00'$
 $E = 1.31'$

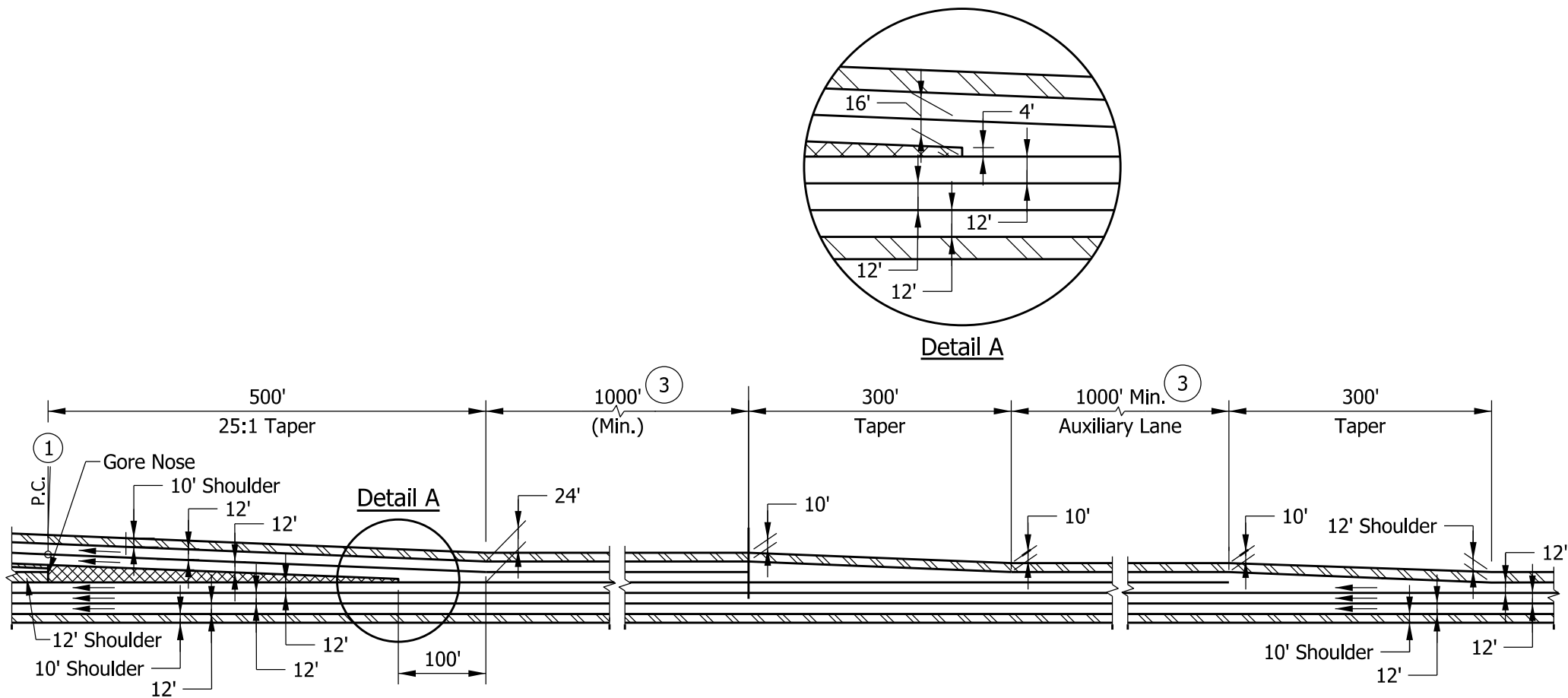


NOTES:

- ① Point of controlling speed on the ramp.
2. See Figure 48-4D for entrance gore details.
- ③ Where truck volumes are high use 70:1 taper.
- ④ The minimum acceleration length "L" is 1000 ft. If the required acceleration length as shown in Figures 48-4H and 48-4I exceed 1000 ft., the additional length shall be added to the length of the parallel lane segment. This length may also be increased contingent on capacity of the downstream freeway segment.
5. For ramps on curves, see Section 48-5.06.

PARALLEL MULTI-LANE ENTRANCE RAMP (SERVICE INTERCHANGE)

Figure 48-4E

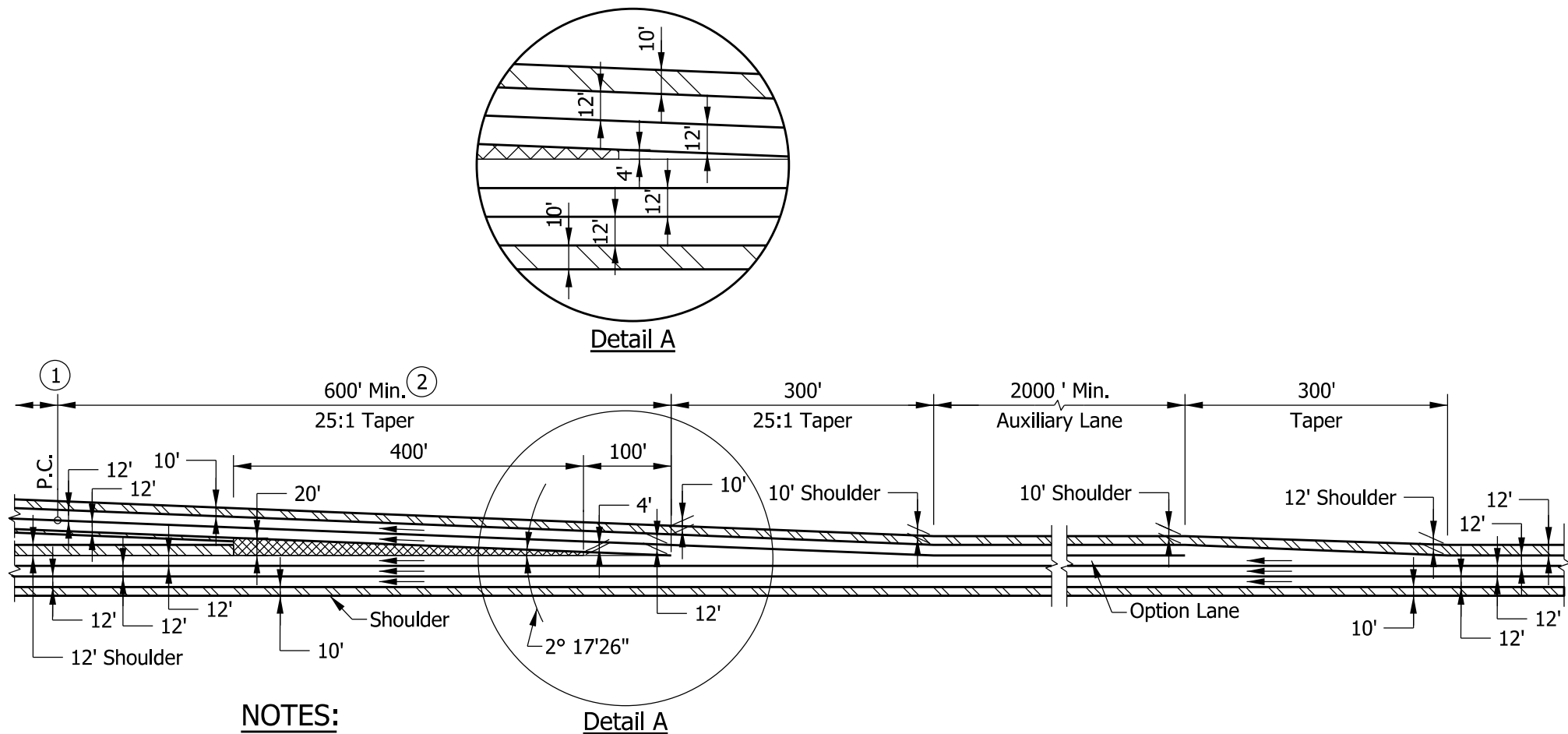


NOTES:

- ① Point of controlling speed at ramp.
2. See Figure 48-4B for exit gore details.
- ③ The designer must coordinate with the Traffic Engineering Corridor Development Office to verify these lengths.
4. For ramps on curves, see Section 48-5.06.

PARALLEL MULTI-LANE EXIT RAMP (SERVICE INTERCHANGE)

Figure 48-4F



NOTES:

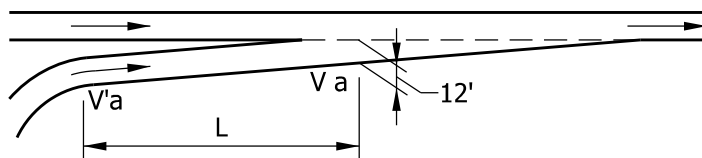
- ① Point of controlling speed at ramp. PC (Point of Curvature) must have minimum design speed complying with middle range for ramp design speed as shown in Figure 48-5A.
- ② See Figure 48-4K and Figure 48-4I for applicable minimum deceleration length and grade adjustments.
- ③ See Figure-48-4D for gore details.
4. The divergence angle must be $2^{\circ}17'26''$ (25:1).
5. For ramps on curves, see Section 48-5.06.

TAPERED MULTI-LANE EXIT RAMP WITH OPTION LANE (SERVICE INTERCHANGE)

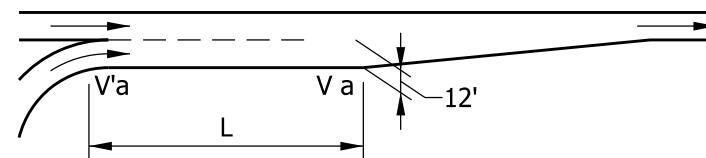
Figure 48-4G

Highway Design Speed (mph)	Speed Reached, V_a (mph)	Acceleration Length, L (ft)								
		for Entrance Curve Design Speed (mph)								
		Stop	15	20	25	30	35	40	45	50
		and Initial Speed, $V'a$ (mph)								
		0	14	18	22	26	30	36	40	44
30	23	180	140	x	x	x	x	x	x	x
35	27	280	220	160	x	x	x	x	x	x
40	31	360	300	270	210	120	x	x	x	x
45	35	560	490	440	380	280	160	x	x	x
50	39	720	660	610	550	450	350	130	x	x
55	43	960	900	810	780	670	550	320	150	x
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

Source: 2011 AASHTO GDHS, Table 10-3
See Figure 48-4I per grade adjustments.



Taper Type



Parallel Type

MINIMUM ACCELERATION LENGTHS FOR ENTRANCE TERMINALS WITH FLAT GRADES OF 3% OR LESS

Figure 48-4H

Acceleration Lanes					
Highway Design Speed (mph)	Ratio of Length on Grade to Length on Level for Design Speed (mph) of Last Ramp Curve				
	20	30	40	50	All Speeds
40 45 50 55 60 65 70	3% ≤ Upgrade < 4%				3% ≤ Downgrade < 4%
	1.30	1.30	--	--	0.700
	1.30	1.35	--	--	0.675
	1.30	1.40	1.40	--	0.650
	1.35	1.45	1.45	--	0.625
	1.40	1.50	1.50	1.60	0.600
	1.45	1.55	1.60	1.70	0.600
	1.50	1.60	1.70	1.80	0.600
40 45 50 55 60 65 70	4% ≤ Upgrade ≤ 6%				4% ≤ Downgrade ≤ 6%
	1.50	1.50	--	--	0.600
	1.50	1.60	--	--	0.575
	1.50	1.70	1.90	--	0.550
	1.60	1.80	2.05	--	0.525
	1.70	1.90	2.20	2.50	0.500
	1.85	2.05	2.40	2.75	0.500
	2.00	2.20	2.60	3.00	0.500
Highway Design Speed (mph)	Deceleration Lanes				
	Ratio of Length on Grade to Length on Level for Design Speed of First Ramp Curve				
All Speeds	>3 to 4 % upgrade 0.9				>3 to 4% downgrade 1.2
All Speeds	>4 to 6 % upgrade 0.8				>4 to 6% downgrade 1.35

- NOTES:**
1. No adjustment is needed for grades of flatter than 3%.
 2. The grade in the table is the average grade measured over the distance for which the acceleration length applies.

Example

Given: Highway Design Speed - 70 mph
Entrance Ramp Curve Design Speed - 45 mph
Average Grade - 4.5% upgrade

Problem: Determine length of acceleration lane.

Solution: Figure 48-4H yields an acceleration length of 820 ft on the level. The grade adjustment for 45 mph is taken as the average of the values for 40 mph and 50 mph. According to the table shown above, this should be increased by the average of the increases shown for 40 mph (2.60) and 50 mph (3.00), or 2.80.

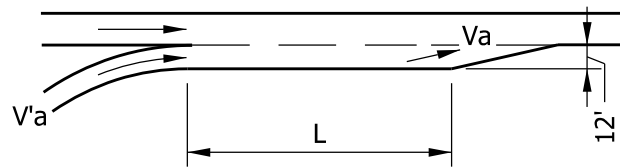
Therefore: $L = (820 \text{ ft})(2.80)$
 $L = 2300 \text{ ft}$

GRADE ADJUSTMENT FOR ACCELERATION/DECELERATION (PASSENGER CAR)

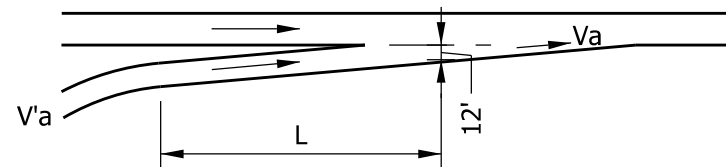
Figure 48-4I

Highway Design Speed (mph)	Speed Reached, V_a (mph)	Acceleration Length, L (ft)								
		for Entrance Curve Design Speed (mph)								
		Stop	15	20	25	30	35	40	45	50
		and Initial Speed, $V'a$ (mph)								
		0	14	18	22	26	30	36	40	44
30	23	275	160	x	x	x	x	x	x	x
35	27	400	300	230	x	x	x	x	x	x
40	31	590	475	400	310	170	x	x	x	x
45	35	800	700	630	540	400	240	x	x	x
50	39	1100	1020	950	850	720	560	200	x	x
55	43	1510	1400	1330	1230	1100	920	580	240	x
60	47	2000	1900	1830	1740	1600	1430	1070	760	330
65	50	2490	2380	2280	2230	2090	1920	1560	1220	800
70	53	3060	2960	2900	2800	2670	2510	2140	1810	1260

Source: NCHRP Report 505, Review of Truck Characteristics as Factors in Roadway Design, Table 66 page 102



Parallel Type



Taper Type

NOTE:

The acceleration lengths are calculated from the distance needed for 180 lb/hp truck to accelerate from the average running speed of the entrance curve to reach a speed (V_a) at 0% grade.

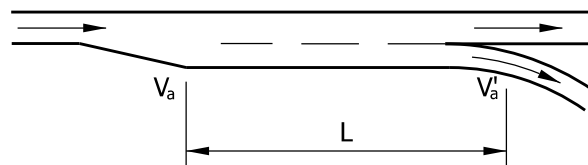
LENGTHS FOR ACCELERATION 180 LB/HP TRUCK

Figure 48-4J

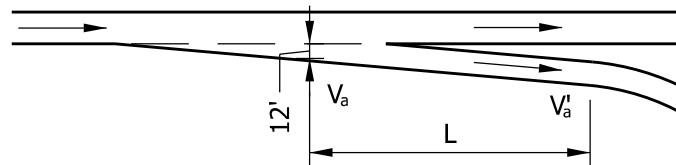
Highway Design Speed (mph)	Speed Reached, V_a (mph)	Deceleration Length, L (ft)								
		for Exit Curve Design Speed of Exit Curve V' (mph)								
		Stop	15	20	25	30	35	40	45	50
		Average Running Speed on Exit Curve, $V'a$ (mph)								
		0	14	18	22	26	30	36	40	44
30	28	235	200	170	140	x	x	x	x	x
35	32	280	250	210	285	150	x	x	x	x
40	36	320	295	265	235	185	155	x	x	x
45	40	385	350	325	295	250	220	x	x	x
50	44	435	405	385	355	315	285	225	175	x
55	48	480	455	440	410	380	350	285	235	x
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

Source: 2011 AASHTO GDHS, Table 10-5

See Figure 48-4I for grade adjustments



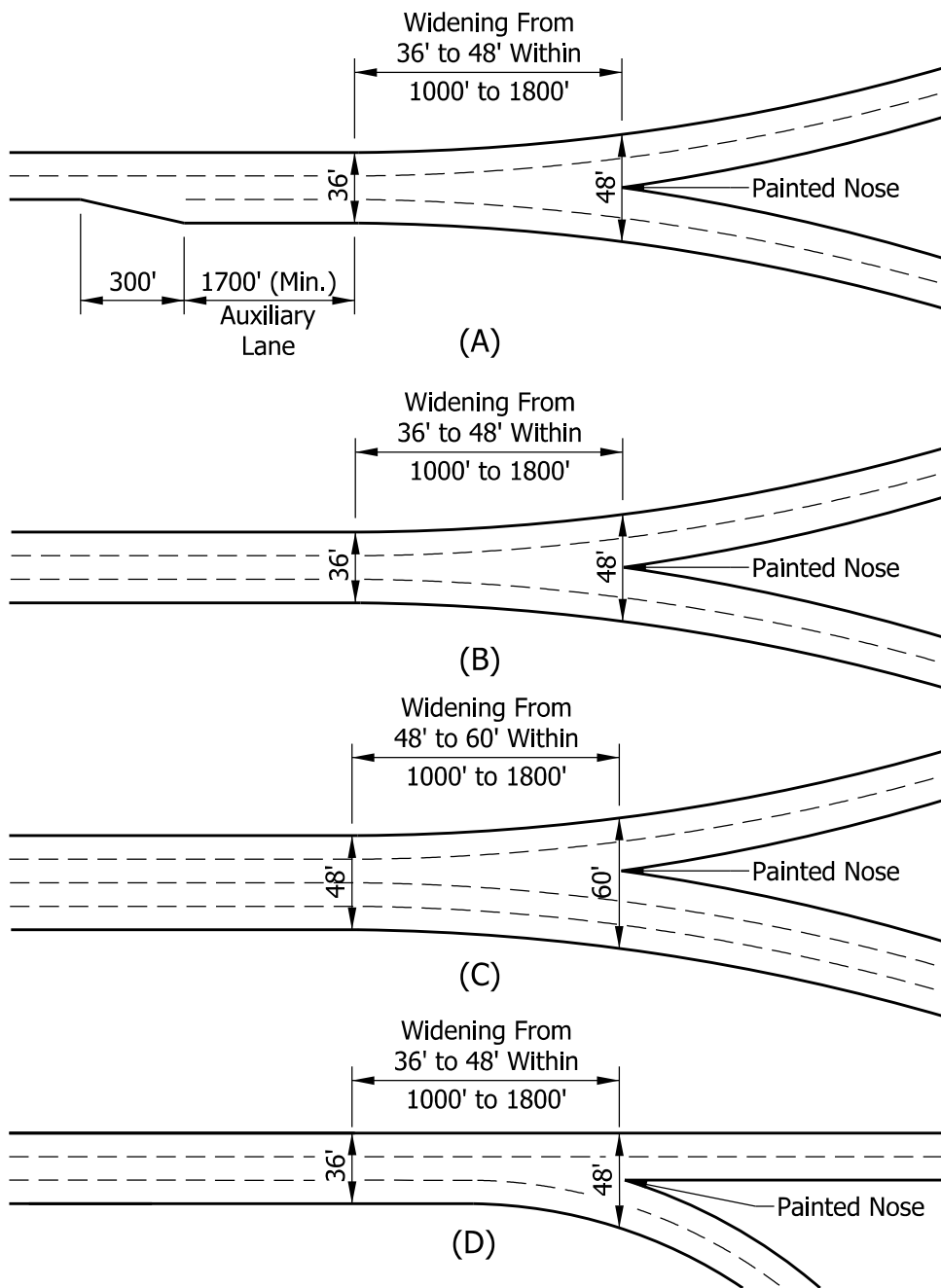
Parallel Type



Taper Type

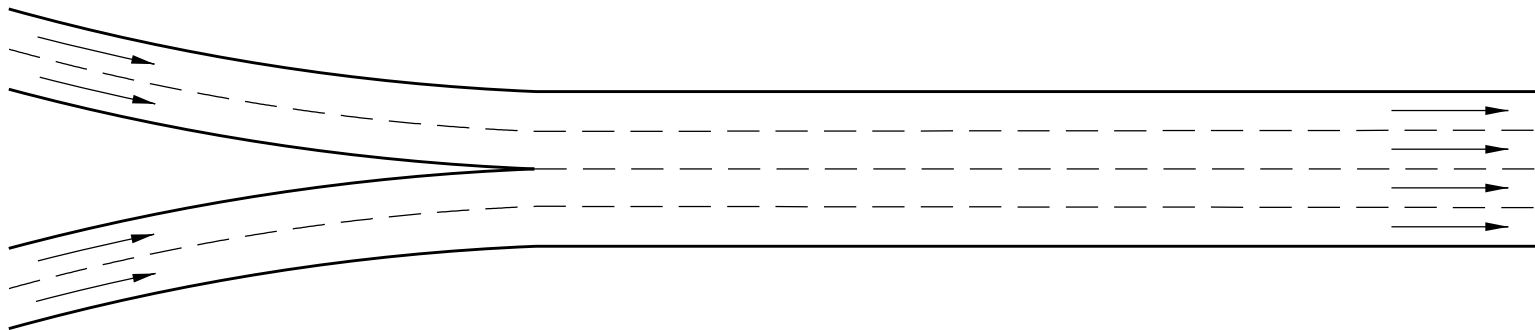
MINIMUM DECELERATION LENGTH FOR EXIT TERMINALS WITH FLAT GRADES OF 3% OR LESS

Figure 48-4K

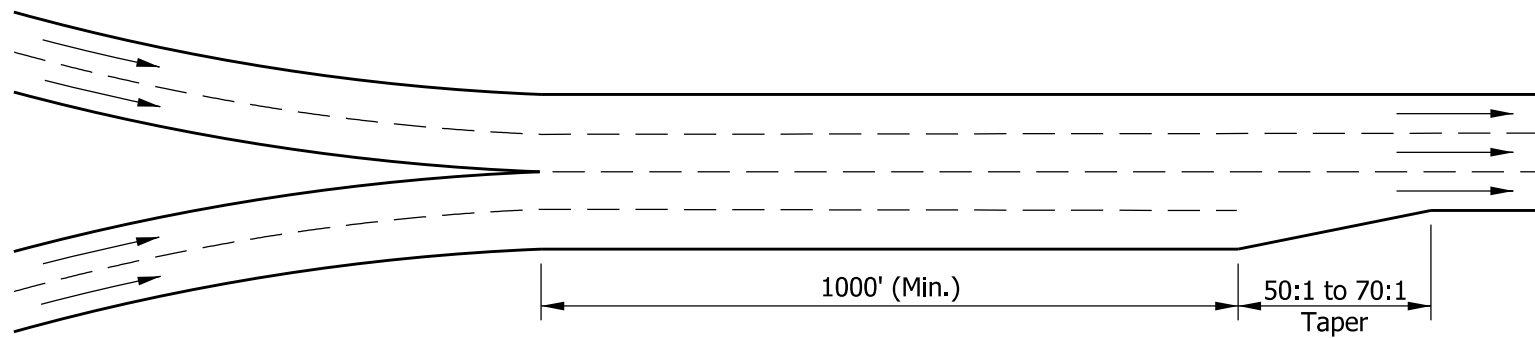


MAJOR FORKS FOR SYSTEM INTERCHANGES (TYPICAL SCHEMATICS)

Figure 48-4L



(A)



(B)

BRANCH CONNECTIONS FOR SYSTEM INTERCHANGES (TYPICAL SCHEMATICS)

Figure 48-4M

Design Element				Manual section	Ramps				
					Single Lane		Multi-Lane		
Interchange Ramp Elements	Travel Lane	Width		48-5.02	16 ft		12 ft per lane		
		Pavement Type (1)		Chapter 304	Asphalt/Concrete				
	Shoulder (2)	Right Width		48-5.02	Usable: 9 ft	Paved: 8 ft	Usable: 11 ft	Paved: 10 ft	
		Left Width			Usable: 5 ft	Paved: 4 ft	Usable: 5 ft	Paved: 4 ft	
		Pavement Type (1)		Chapter 304	Asphalt/Concrete				
	Cross Slope (3)	Travel Lane		48-5.02	2%				
		Shoulder			Right: 4% Left: 2%				
	Superelevation				48-5.03	4%, 6%, or 8%			
	Clear Zone Width				49-2.0	(4)			
	Side Slopes	Cut	Foreslope		45-3.0	6:1 (5)			
			Ditch Width			4 ft (6)			
			Backslope			4:1 (7)			
		Fill		6:1 to Clear Zone: 3:1 max to Toe					

- (1) Pavement Type. The pavement selection will be determined by the Office of Pavement Engineering.
- (2) Shoulder Width. Shoulder width criteria apply regardless of the presence of curb. Curbs should be considered only to facilitate drainage needs. Only sloping curb should be used.
- (3) Cross Slope (Shoulders). For ramps on a curve, the shoulder cross slope will be the same as the traveled way. See Section 48-5.03.
- (4) Clear Zone. This will vary according to design speed, traffic volume, side slopes, and horizontal curvature.
- (5) Foreslope. See Section 49-2.0 and 49-3.0 for the lateral extent of the foreslope in a ditch section.
- (6) Ditch Width. A "V" ditch should be used in a rock cut.
- (7) Backslope. For an earth cut of 10 ft or deeper, the first horizontal 20 ft of the backslope will be sloped at a rate of 4:1. Then, a slope rate of 3:1 is normally used to the natural ground line. The backslope for a rock cut will vary according to the height of cut and the geotechnical requirements. See Sections 45-3.0 and 107-6.01.

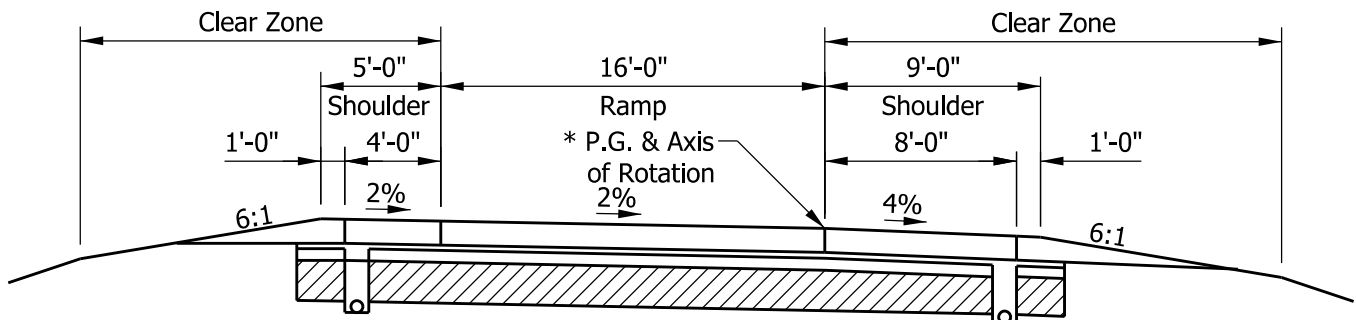
Highway Design, Speed, mph*	40	45	50	55	60	65	70
	Ramp Design Speed, mph						
Upper Range, 85%	35	40	45	45	50	55	60
Middle Range, 70%	30	35	35	40	45	45	50
Lower Range, 50% (Loop Ramps Only)	20	25	25	30	30	30	35

* Source: 2011 AASHTO GDHS, Table 10-1 Only highway design speeds of 50 mph or higher may be applied to a freeway or expressway exit.

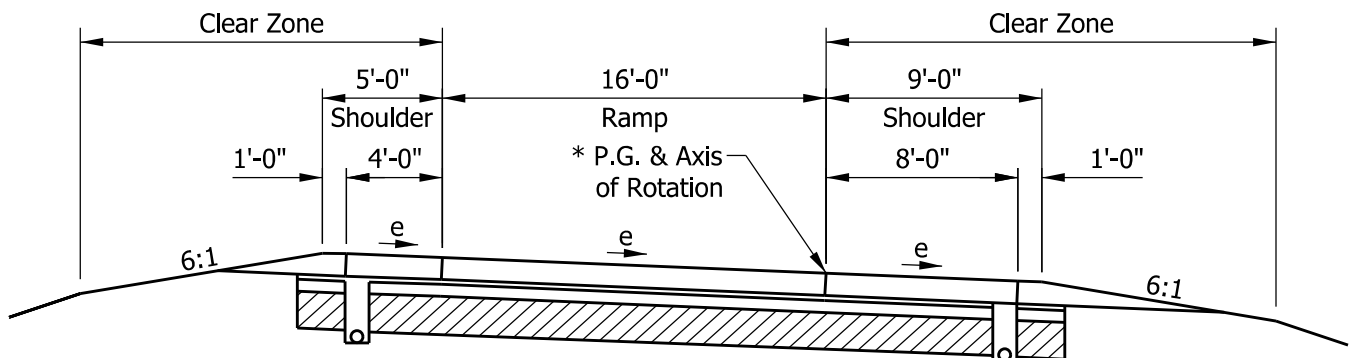
Ramp Design Speed

RAMP GEOMETRIC AND DESIGN CRITERIA

Figure 48-5A



Single Lane Ramp Section in Tangent

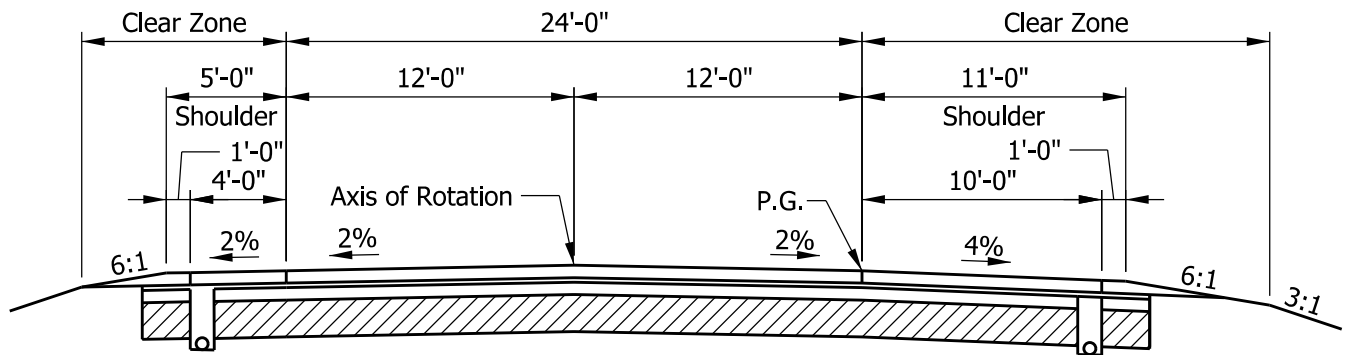


Single Lane Ramp Section in Superelevation

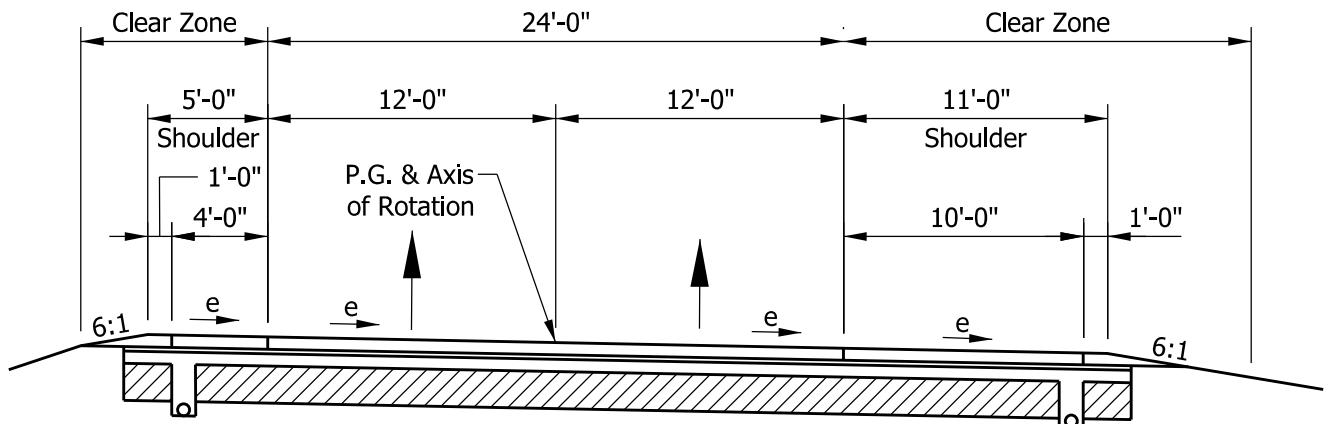
* The axis of rotation and PG may be moved to the centerline of the ramp to reduce transition length.

SINGLE LANE RAMP TYPICAL SECTION

FIGURE 48-5B



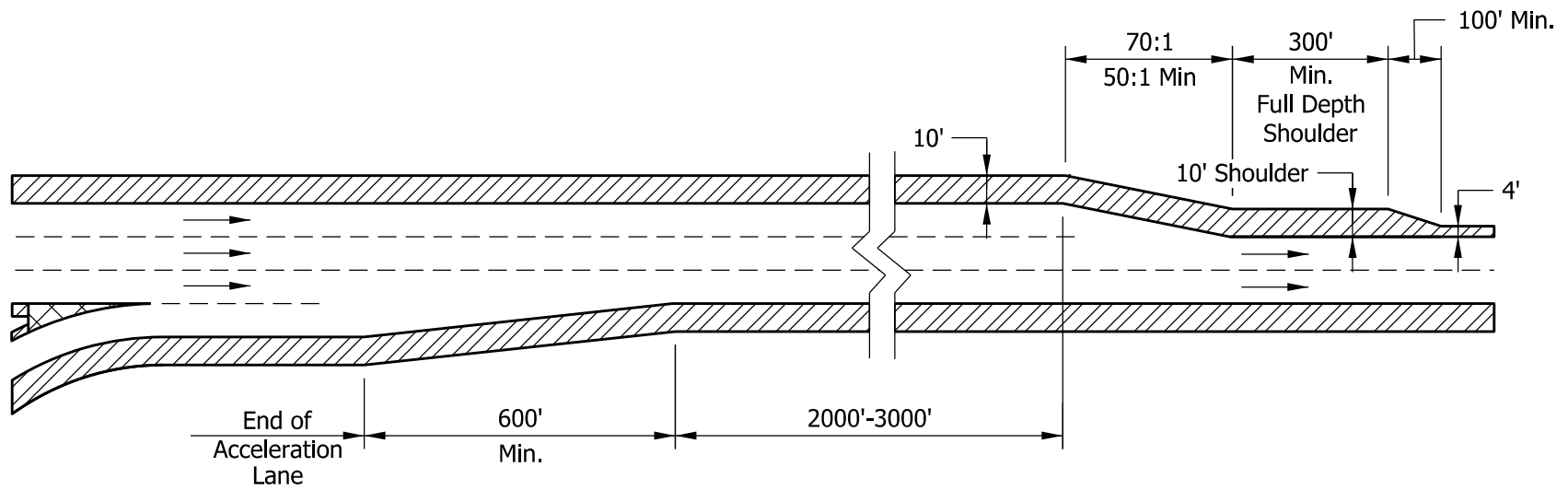
Multi-Lane Ramp in Tangent



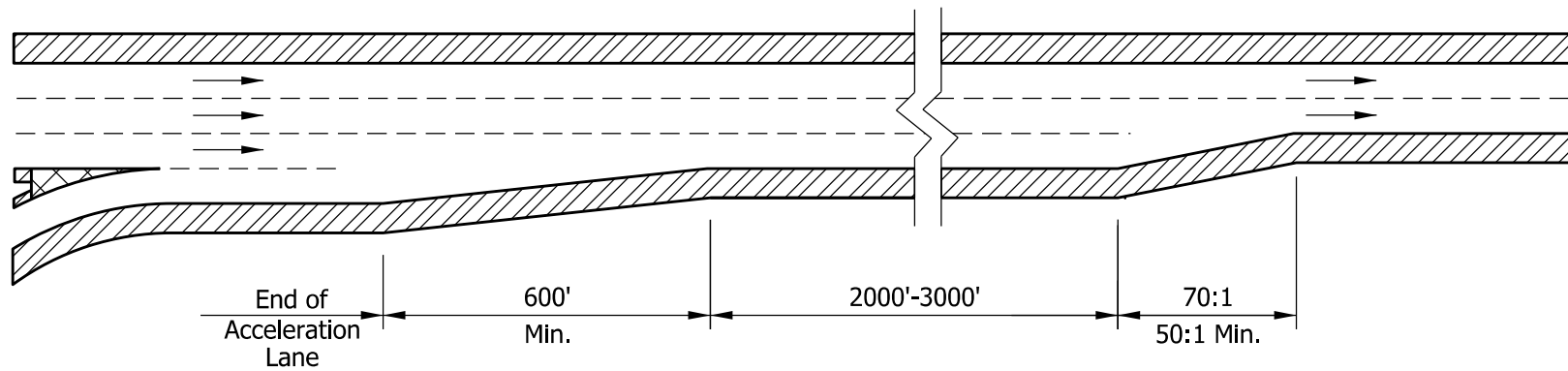
Multi-Lane Ramp in Superelevation

MULTI-LANE RAMP TYPICAL SECTION

Figure 48-5C



Left Side Freeway Lane Drop



Right Side Freeway Lane Drop

FREEWAY LANE DROP

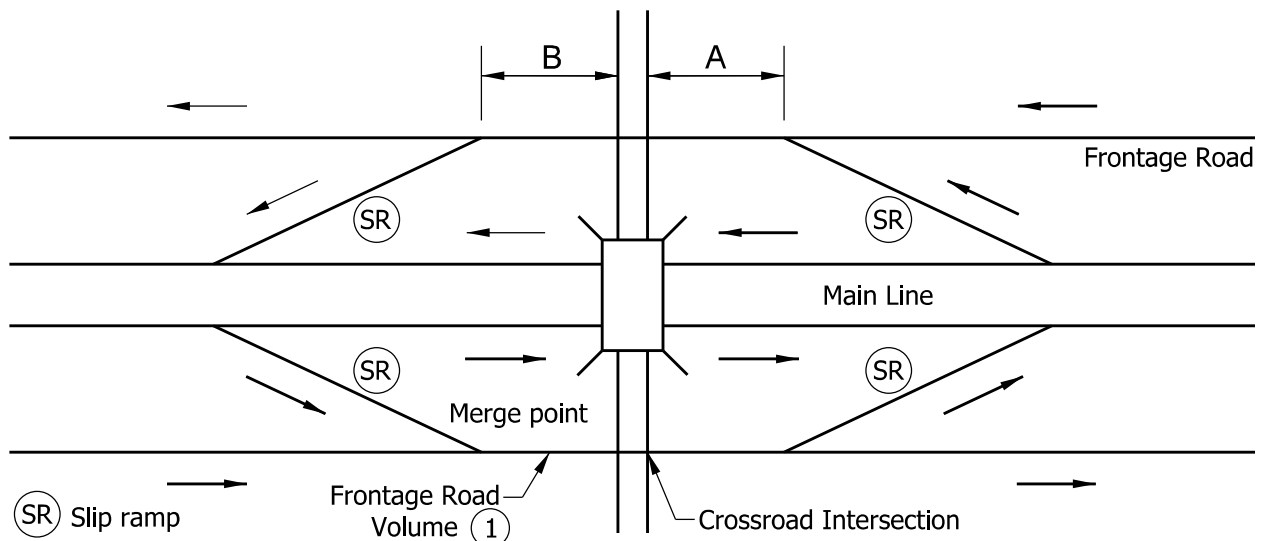
Figure 48-6A

Frontage Road Volume (vph) (2)	Exit Ramp Volume (vph) (3)	A (ft) (4)		
		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	460
1200	830	640	870	480
1400	960	690	970	500
1600	1100	770	1070	530
1800	1240	860	1180	560
2000	1380	970	1300	590

Source: Transportation Research Record 682, Table 3

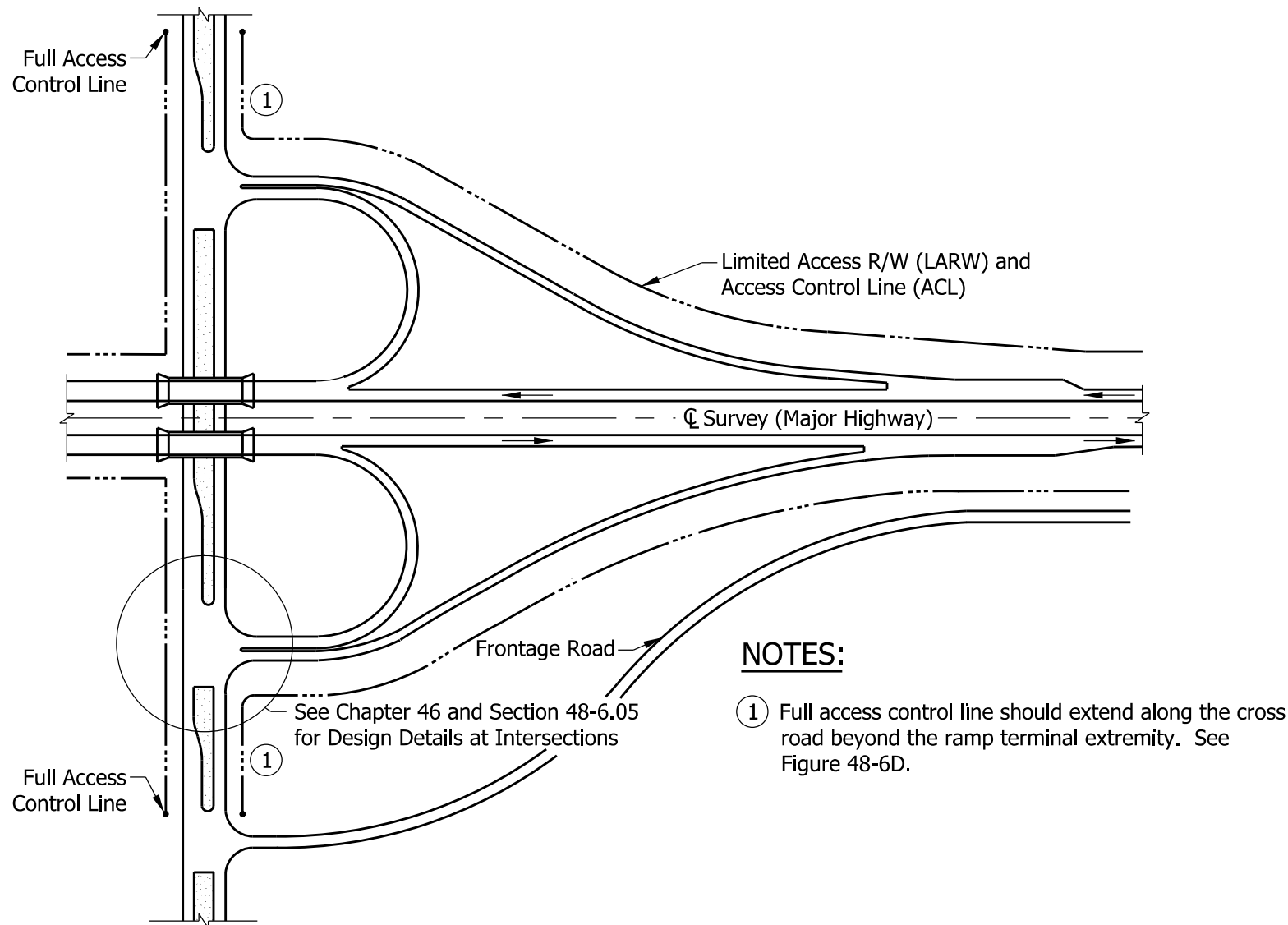
NOTES:

1. Table values are acceptable for planning purposes only. Final dimensions will be based on a detailed operational analysis. This design may only be used where necessary in a restricted urban area.
- (2) Total frontage road and exit ramp traffic volume between merge point to intersection with minor road.
- (3) Assumed to be 69% of total volume shown in first column.
- (4) Distance "A" is from the Exit Ramp center-line point of merge with the frontage road to the signalized intersection.
5. Distance "B" is determined on a case by case basis.



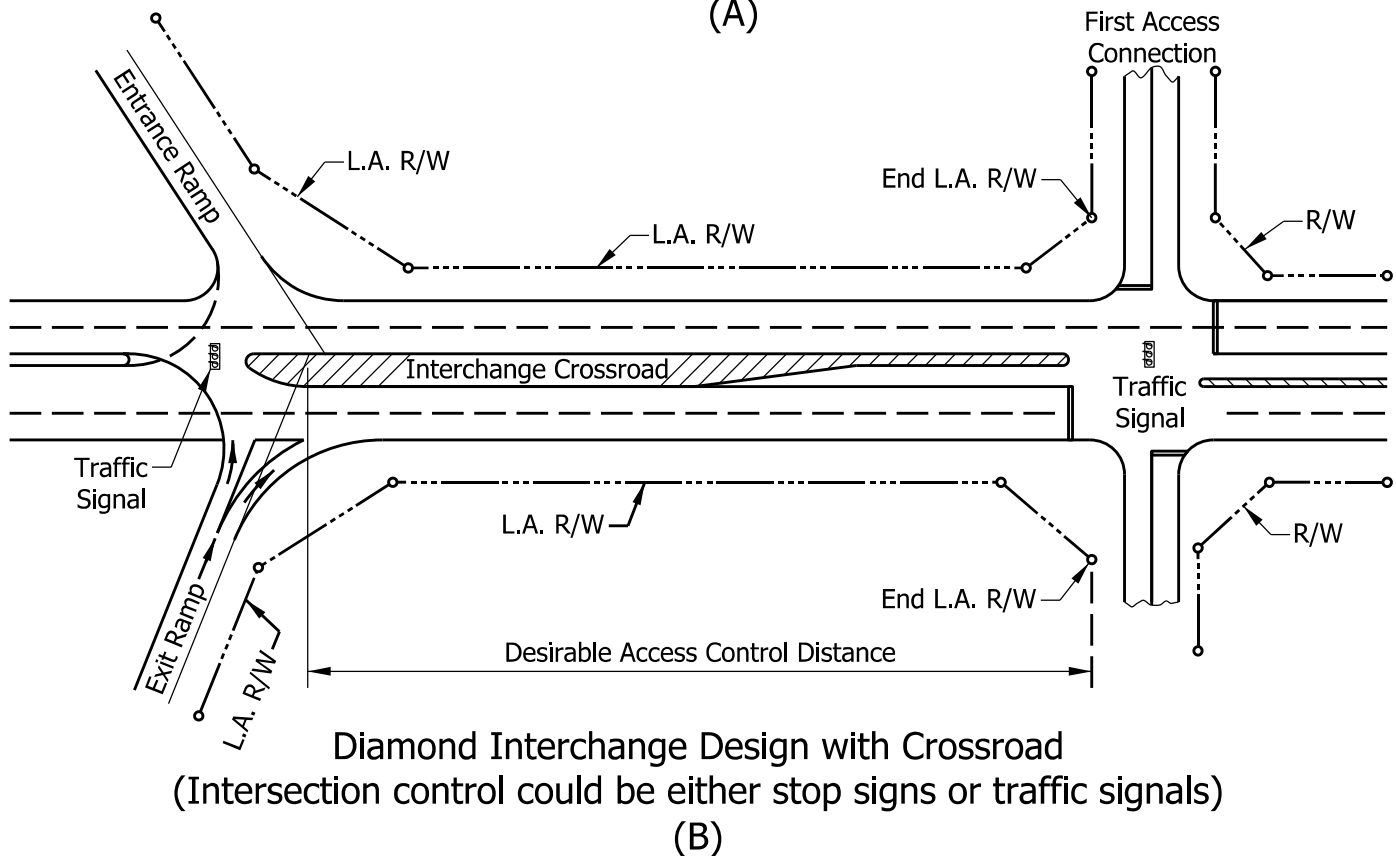
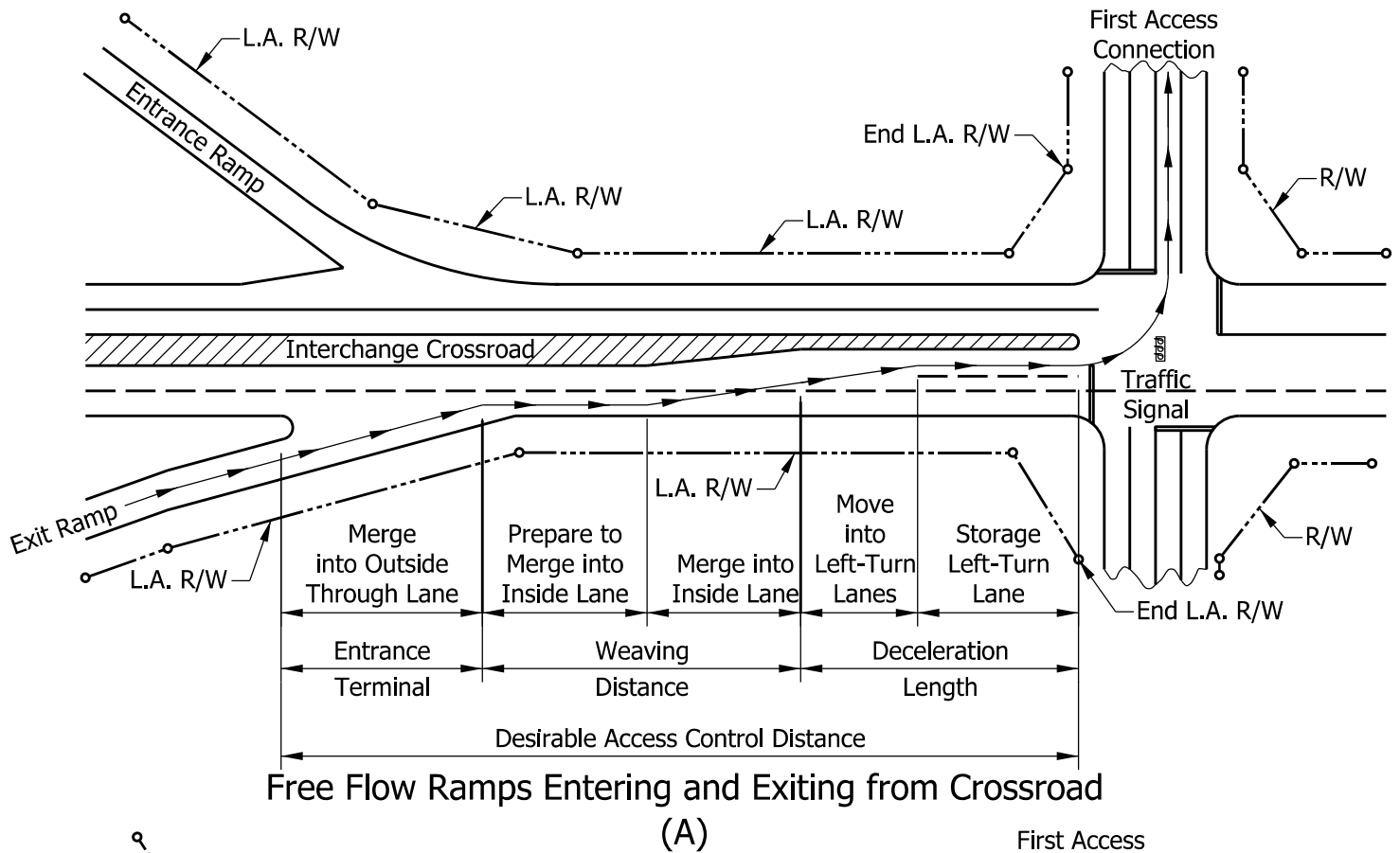
RAMP AND CONTINUOUS FRONTAGE ROAD INTERSECTION (For Planning Only)

Figure 48-6B



TYPICAL ACCESS CONTROL FOR A PARTIAL CLOVERLEAF INTERCHANGE

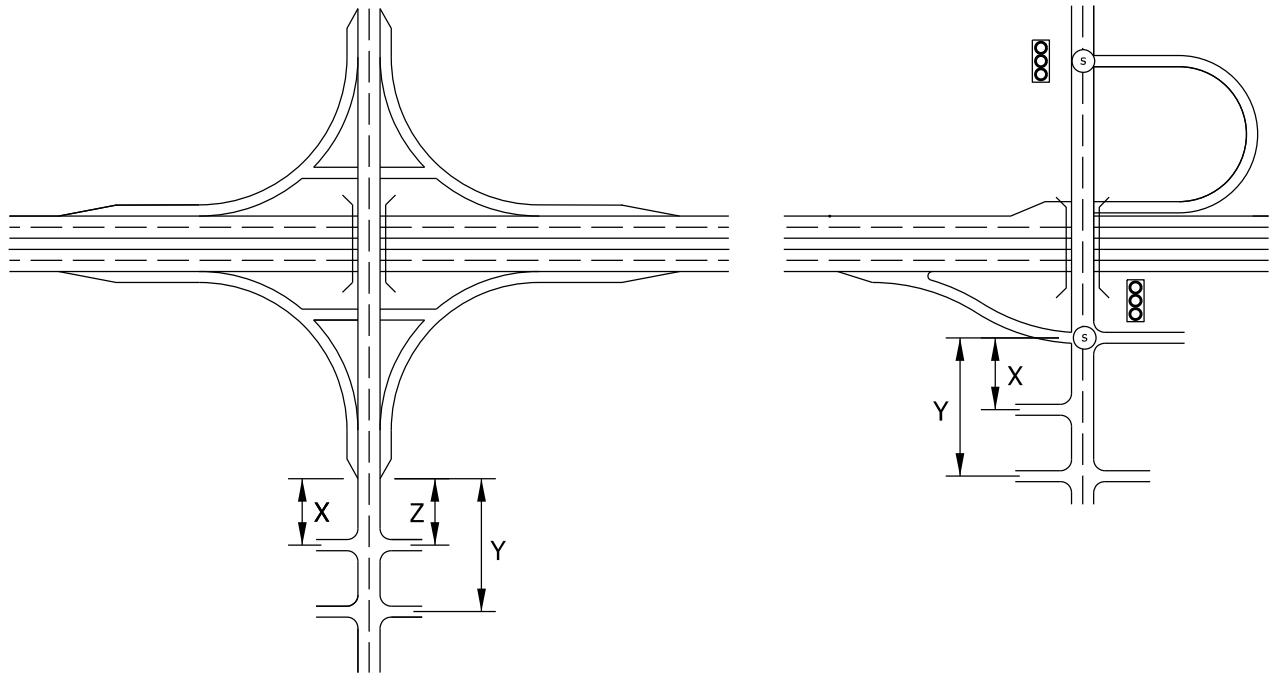
Figure 48-6C



ACCESS CONTROL AT RAMP TERMINALS

Figure 48-6D

Minimum Spacing for Intersections and Commercial Entrances Near Interchange Area on Two-Lane Crossroads		
	X or Z	Y
Urban	600'	1000'
Rural	750'	1320'



X = Distance to first entrance on the right from end of off-ramp terminal.

Y = Distance to first four legged intersection measured from the end of the off-ramp terminal or from the start of the terminal for the on-ramp.

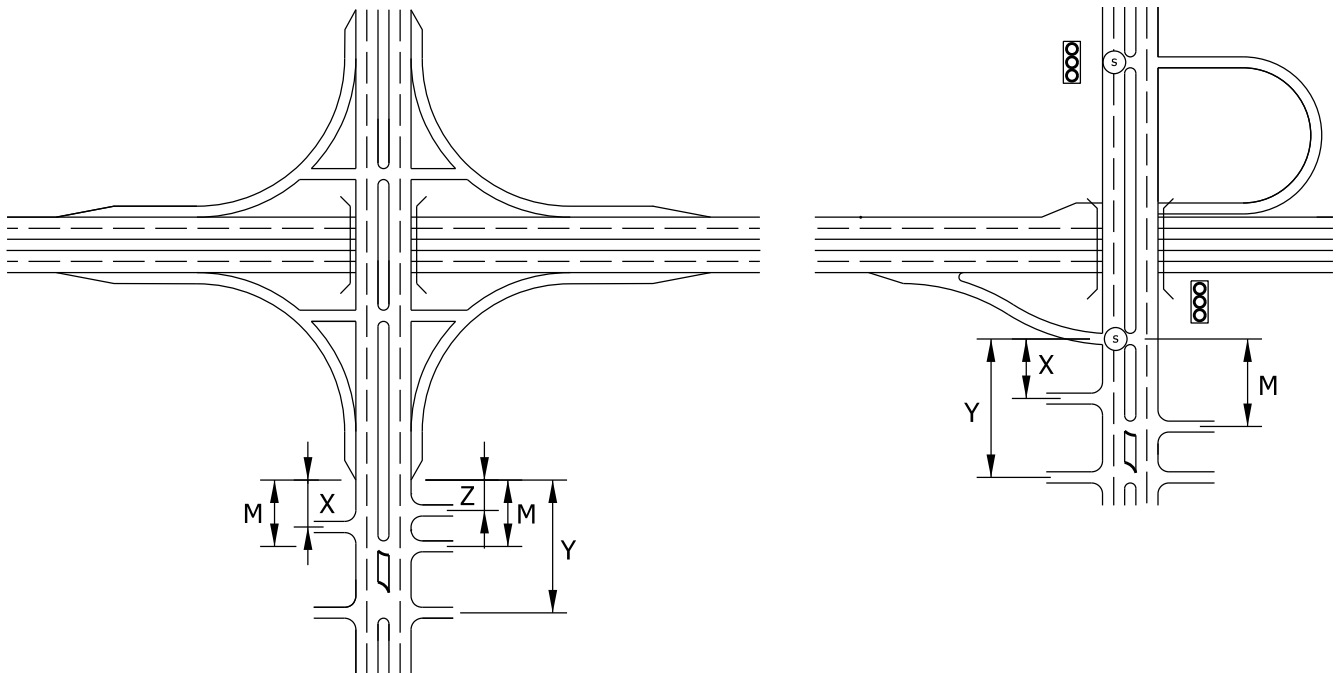
Z = Distance between the last entrance connection and the start of the terminal for the on-ramp.

Note: Spacing applies to both signalized and unsignalized intersections and commercial entrances regardless of the interchange configurations.

ACCESS CONTROL ON TWO-LANE CROSSROADS AT INTERCHANGES

Figure 48-6E

Minimum Spacing for Intersections and Commercial Entrances Near Interchange Areas on Multi-Lane Crossroads		
X or Z	Y	M
600'	1320'	1000'



M = Distance to the first directional median opening or no full median openings are allowed in non-traversable medians up to the first major intersection.

X = Distance to first entrance on the right from end of off-ramp terminal.

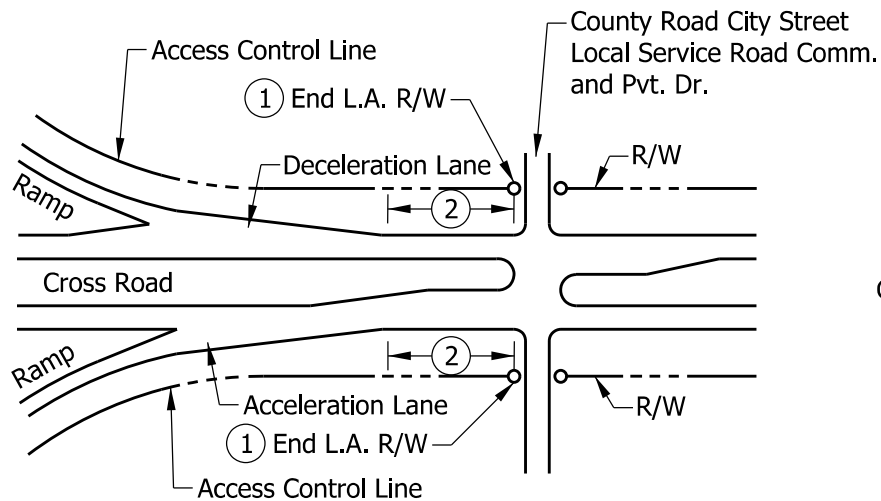
Y = Distance to first four legged intersection measured from the end of the off-ramp terminal or from the start of the terminal for the on-ramp.

Z = Distance between the last entrance connection and the start of the terminal for the on ramp.

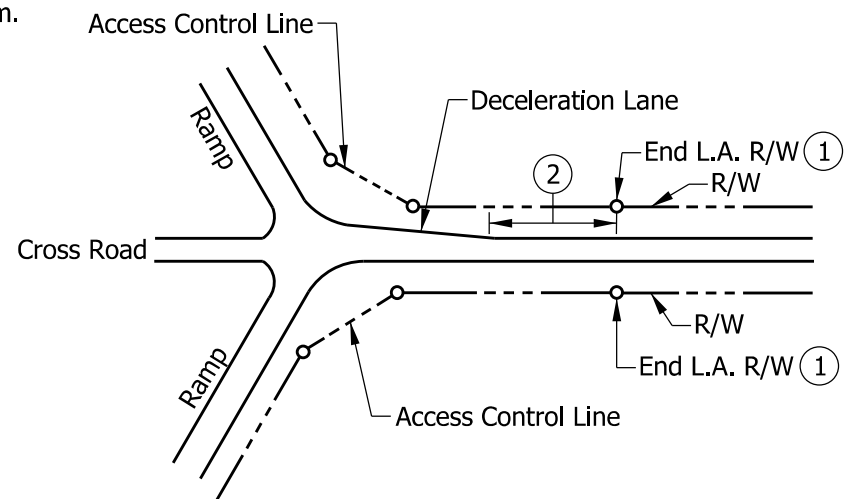
Note: Spacing applies to both signalized and unsignalized intersections and commercial entrances regardless of the interchange configuration.

ACCESS CONTROL ON MULTI-LANE CROSSROADS AT INTERCHANGES

Figure 48-6F



(A) Multi-Lane Roadway



(B) Two-Lane Roadway

NOTES:

- ① Full access control line should extend along the cross road beyond the ramp terminal taper extremity (both sides of road) minimum of 600 ft in urban areas and 750 ft in rural areas. The end of access control should be at opposite points, where feasible.
- ② The auxiliary lane terminating the greater distance from the interchange area governs.

LIMITED ACCESS RIGHT OF WAY AT RAMP TERMINALS

Figure 48-6G