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CHAPTER FORTY-EIGHT

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 Procedure

The Office of Environmental Services' Environmental Policy Team is responsible for determining the need for, location of, and type of interchange. This assessment is based on a consideration of factors which are discussed in Sections 48-1.0 and 48-2.0. The designer is responsible for determining the layout and design of an 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 consideration of its benefits. Because of the great variance in specific site conditions, INDOT has not adopted specific interchange warrants. If determining the need for an interchange or grade separation, the following should be considered.

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 a facility with partial control of access, an intersection with a public road will be accommodated with an interchange or with an at-grade intersection. A grade separation alone is not normally provided. An interchange will be selected for a higher-traffic-volume intersecting road. Therefore, on a facility with partial control of access, the decision to provide an interchange will be based on the criteria described 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 are the most tangible interchange guidelines, The Department has not adopted specific levels which, if exceeded, would demand an interchange. On a facility 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 a railroad-highway crossing should be considered. Section 48-3.08 provides additional information on safety considerations relative to interchange selection.

4. Site Topography. The topography may be more adaptable to an interchange than an at-grade intersection.

5. Road-User Benefits. An interchange significantly reduces the travel time if compared to an at-grade intersection but may increase travel distance. If an analysis reveals that road-user benefits over the service life of the interchange will exceed costs, an interchange may be considered. For more information on road-user benefit analysis, see Chapter Fifty.

6. Traffic Volume. An interchange should be considered at a crossroad with high traffic volume because elimination of conflicts greatly improves the movement of traffic.

7. Other Factors. Other factors 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 route is considered to be 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 considered to be 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 to be a revised access. Lengthening or adding an auxiliary lane at an at-grade ramp terminal with a crossroad or ramp-proper lane is not, nor is converting a single-lane off- or on-ramp to two 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's *A Policy on Design Standards – Interstate System*, and this *Manual*.

Work determined to consist of new or revised access to the existing Interstate System will require development by INDOT to FHWA of a formal Request for New or Revised Access to the Interstate System, commonly referred to as an Interstate Justification (IJ) Study Report. The IJ is a stand-alone document which constitutes a request from INDOT for FHWA approval of new or revised access. The document will demonstrate that reasonable care has been taken in addressing eight criteria described in the *Federal Register* of February 11, 1998, and Section 48-1.03(03), confirming that future traffic operations along the affected Interstate corridor will not be adversely affected by the proposed action. The entire Interstate System in the State is under jurisdiction of INDOT. Only the Department, and not a local public agency or private concern, may develop an IJ and submit it to FHWA for approval.

The requirement for an IJ and such FHWA approval applies only for a non-tolled Interstate route or Interstate toll road where federal-aid funds have been expended or where a tolled section have been added to the Interstate System under the requirements of 23 USC 139(a). Access to a non-Interstate freeway or to a new Interstate highway does not require an IJ. The Department has the authority to approve new or revised access to another type of route where federal-aid funds were used to acquire the access control. For this situation, the Department 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. The Department may request FHWA advice or assistance on the acceptability of this type of new or revised access if desired.

48-1.03(02) Actions Requiring an IJ

The actions that require Department development and FHWA approval of an IJ are as follows:

1. establishing a new freeway-to-freeway (system) interchange;

2. major modification of a freeway-to-freeway 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. establishing a new or revised partial interchange of any form;

4. establishing a new freeway-to-non-freeway (service) interchange;

5. modification of an existing freeway-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) elements;

6. removal from service of a select access point or ramp or an entire interchange;

7. changing the essential type of interchange, e.g., replace conventional diamond with partial cloverleaf;

8. changing the essential form of a ramp, e.g., directional, semi-directional, loop, or diagonal;

9. 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 Department personnel and not the general public; or

10. other form of new or revised access not explicitly listed above, e.g., that rising to a level beyond incidental work.

48-1.03(03) Actions Not Requiring an IJ

The actions that do not require development of an IJ are as follows:

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. minor horizontal or vertical realignment of a ramp;

5. converting a taper-type on- or off-ramp to one of a parallel-type;

6. increasing the length of an on-ramp acceleration lane or an off-ramp deceleration lane;

7. addition of one or more continuous auxiliary lanes between two adjacent interchange ramps; or

8. other minor action not explicitly listed above.

An analysis of traffic operation should be conducted. The Department should informally consult with the appropriate FHWA Transportation Engineer even if such project is not subject to FHWA oversight.

48-1.03(04) Coordination with National Environmental Policy Act (NEPA) Requirements

If a federal agency is required to make an approval action, regardless of the funding source, the NEPA process must be followed. Therefore, since FHWA approves from INDOT, a formal Request for New or Revised Access to the Interstate System (IJ analysis), the NEPA process must be followed if developing new or revised Interstate access. The NEPA process should proceed concurrently with development and analysis of (existing) Interstate access alternatives to ensure that all decision-making regarding all viable alternatives that are expected to be acceptable by FHWA from a traffic-operations standpoint are analyzed and adequately considered. FHWA final IJ approval can only be obtained after completion of the NEPA process. 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 to be made as part of the NEPA process.

48-1.03(05) General Steps in Revising or Adding Access to the Interstate System

There are five major steps that should be followed for alternatives' development of IJ development for a more-complex proposed new or revised access to the Interstate System. 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 IJ process. Not all of these decision points are necessary for IJ development for a less-complex proposed new or revised access. In coordination with the appropriate FHWA Project Management Team Leader, some or all of the early decision points may be determined to be unnecessary and that only final approval should be requested. The basic steps, or decision points, are as follows.

1. <u>Development of Alternatives</u>. At the start of alternatives' development for an action that may ultimately require IJ preparation and approval, the Department will meet with FHWA to identify special process and operational requirements. During the Engineering Assessment phase and early in the NEPA process, one or more alternative functional designs should be examined from primary aspects of traffic operation, safety, and cost-

effectiveness in concert with overall social, economic, and environmental consequences. Alternatives that would not function adequately from a safety or traffic-operations standpoint should be eliminated. During the NEPA alternatives'-screening process, appropriate intensity-of-alternatives' development should be carried out, along with analysis and coordination with other parties having a stake in the screening and ultimate access decision. The Production Management Division's Environmental Policy Team oversees development of IJ activities. The appropriate FHWA Project Management Team will serve as the Department's point of contact for this process of developing and screening alternatives. The Team's Transportation Engineer will represent FHWA in providing opinion and review of alternatives from a transportation-operations standpoint.

2. Concept Approval. A letter requesting concept approval of a new or revised access element will be submitted to FHWA once a single alternative has been identified as the conditionally recommended course of action emerging from the access concept's development phase and ongoing NEPA process. This may occur either before the Draft EIS is approved or before the final EIS, EA, or Categorical Exclusion (CE) is approved. If appropriate, the FHWA Project Management Team leader will respond in writing within two weeks indicating the acceptability in concept of the recommended alternative and allow for the completion of the appropriate NEPA documentation and preparation of the formal IJ request. This will represent FHWA's Concept Approval, and is FHWA's opinion with respect to the engineering and operational acceptability of the recommend alternative based on the information available at that time. FHWA's Concept Approval is given with the understanding that the proposal will be that which is reflected in the final NEPA document; CE, Finding of No Significant Impact (FONSI), or Record of Decision (ROD).

3. Draft IJ Report Development. The Department will initiate a meeting with FHWA to determine the scope of assessment unique to the particular new or revised access element. The Department will then prepare the draft document, focusing on the eight points of the *Federal Register* of February 11, 1998. The draft IJ will be submitted to the FHWA for comments.

4. Final IJ Submittal. Upon written reply and comments on the draft IJ from FHWA, the necessary revisions should be made. The Department may meet with FHWA to resolve significant issues, or upon request from FHWA. The final IJ should not be forwarded to FHWA until the preferred alternative within the context of the NEPA process is identified. By cover letter with the final IJ, the Department will request from FHWA a determination of engineering and operational acceptability of the new or revised access. The letter will also include the status of the NEPA evaluation.

5. Provisional and Final IJ Approval. FHWA will respond in writing within four weeks to INDOT's formal request for approval of new or revised access, effectively approving the

final IJ. The letter from FHWA will indicate approval or denial of the request. It is understood that approval of the IJ proposal is provisional, if at that stage the NEPA process has not been fully executed. Upon approval of the final environmental document (CE, FONSI, or ROD), FHWA will issue the Department final IJ approval in writing.

48-1.03(06) Content of the IJ

The Request for New or Revised Access to the Interstate System, or IJ, must address the eight criteria outlined in the *Federal Register* of February 11, 1998, and described below. These criteria will be the focus of attention in the IJ. The IJ must directly respond to the eight criteria, in the order shown below. Other background information may be provided to supplement that core element. A clear description of the proposed new or revised access should be provided, generally in narrative form directing the reader to sketch-plan drawings. All relevant notes, summary printouts, or electronic input/output files of traffic operations analysis should be appended to the IJ document, be they from HCM / HCS, 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 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. Necessary design exceptions should desirably be identified. 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 the Department.

The following lists and clarifies the criteria shown in the *Federal Register* of February 11, 1998. For each of the eight criteria, the first paragraph restates the language in the *Federal Register*, unedited. The subsequent paragraphs serve to clarify the core statement.

1. <u>Existing Facilities</u>. *The existing interchanges and/or local roads and streets in the corridor can neither provide the necessary access nor be improved to satisfactorily accommodate the design year traffic demands while at the same time providing the access intended by the proposal.*

   The IJ should demonstrate that an access point is needed for regional traffic needs and not to solve local transportation needs. It is of utmost importance to maintain the integrity and primary function of the Interstate System. The Interstate facility should not be permitted to become part of the local circulation system but should be maintained as the main regional and inter-state highway it was intended to be. All reasonable measures should be made to provide local access and mobility by means of the non-Interstate network.

Existing or possible future roads or streets in the vicinity of the Interstate facility should be evaluated or considered for use as connections to existing adjacent interchange ramps, in lieu of adding a new interchange or ramp(s).

2. *Transportation System Management (TSM). All reasonable alternatives for design options, location, and transportation system management type improvements (such as ramp metering, mass transit, and HOV facilities) have been assessed and provided for if currently justified, or provisions are included for accommodating such facilities if a future need is identified.*

   All TSM strategies, including those that involve improvements to existing non-Interstate roads and streets, should be fully explored in lieu of new or revised access to the Interstate system.

3. *Access Connections and Design. The proposed access connects to a public road only and will provide for all traffic movements, except in only the most extreme circumstances. Less than full interchanges for special purpose access for transit vehicles, for HOVs, or into park and ride lots may be considered on a case-by-case basis. The proposed access will be designed to meet or exceed current standards for federal-aid projects on the Interstate System.*

   Except in the most extreme circumstance, each interchange should provide for all basic movements. A partial interchange is generally unacceptable, in part because it has undesirable operational characteristics. Private-road access is not permitted on the Interstate System.

4. *Transportation Land Use Plans. The proposal considers and is consistent with local and regional land use and transportation plans. Prior to final approval, all requests for new or revised access must be consistent with the metropolitan and/or statewide transportation plan, as appropriate, the applicable provisions of 23 CFR 450 and transportation conformity requirements of 40 CFR 51 and 93.*

   Coordination with strategic, long-term transportation plans should be ensured, so as not to have fragmented consideration of revised or added access. The IJ should include a discussion as to how the proposal fits into the overall transportation plans for the area and, if it is an addition to the current plans for the area, how it affects the current plans. The IJ proposal does not have to be included in an official transportation plan or be approved by a metropolitan planning organization (MPO) or similar organization prior to submittal to FHWA. However, if the project is within an MPO area, coordination with the MPO must occur. All such coordination must be completed before FHWA approval of the IJ. This should form part of the normal project-development process. The expectation here is that any proposal is considered in view of currently-known plans for transportation facilities or land use planning.

5. *Comprehensive Interstate Network Study*. *In areas where the potential exists for future multiple interchange additions, all requests for new or revised access are supported by a comprehensive Interstate network study with recommendations that address all proposed and desired access within the context of a long-term plan.*

   To the extent practicable, the Department will program and thus allow coordinated analysis and project development of logical Interstate segments which may include multiple access sites (interchanges).

6. *Coordination with Transportation System Improvements*. *The request for a new or revised access generated by new or expanded development demonstrates appropriate coordination between the development and related or otherwise required transportation system improvements.*

   It is incumbent upon the Department and FHWA to ensure that the Interstate System is preserved and improved in an orderly and coordinated manner to serve the public and maintain the essential function of this most important network of national highways. Therefore, if private development is the impetus behind the need for access, it is necessary to coordinate efforts with the private party in order to develop the access to achieve mutual benefits with no safety or operational adverse impacts on Interstate-route users.

7. *Status of Planning and NEPA*. *The request for new or revised access contains information relative to the planning requirements and the status of the environmental processing of the proposal.*

   Information should be confirmed and reported relative to the status of the planning and NEPA processes with regard to the access request.

8. *Operational Analysis. The proposed access point does not have a significant adverse impact on the safety and operation of the Interstate facility based on an analysis of current and future traffic. The operational analysis for existing conditions shall, particularly in urbanized areas, include an analysis of sections of Interstate to and including at least the first adjacent existing or proposed interchange on each side. Crossroads and other roads and streets shall be included in the analysis to the extent necessary to assure their ability to collect and distribute traffic to and from the interchange with the new or revised access points.*

Sufficient operational analyses should be made to determine the impact of the revised or new access on the Interstate-route operation. The Transportation Research Board's *Highway Capacity Manual (HCM)* analysis procedures should be used. Analysis based on other methodologies is not acceptable. The *HCM*'s companion software, HCS, may be used. Other software tools that precisely replicate *HCM* methodologies may be used. Analysis by means of other (software) models that do not precisely employ *HCM* equations and logic may be presented but only as supplementary information.

The operational analysis should be extended as far along the mainline and should include adjacent downstream interchanges as necessary to establish the extent and scope of the impacts. This could be critical in an urban area with many interchanges spaced at less than 1.6 km apart. As a minimum, the operational impact on the mainline Interstate route between the proposed new or revised access and immediately adjacent existing downstream interchanges on either side must be analyzed. The exact adjacent interchanges to be analyzed will be determined jointly by FHWA and the Department. Crossroad analysis is always required at the subject (core) interchange, between, through, and outside of ramp terminals on the crossroad. Analysis of the crossroads of the adjacent downstream interchanges is normally not required in an IJ, unless circumstances dictate otherwise.

Appropriate, sanctioned traffic data provided by the Planning Division's Traffic Monitoring Team should be used as the basis for operational analysis for the IJ process. The traffic counts and projections should be approved by the Department, developed using acceptable industry and agency standards.

a. Drawings. A dimensioned drawing(s) of preferred scale 1:2000 to 1:4000 should be provided as an attachment to the IJ document. The drawing(s) should show the functional elements of the existing and proposed conditions including, as applicable, project limits, adjacent interchange(s) along the freeway, adjacent intersections along the crossroad, ramps to be added, ramps to be removed, relocation of ramp gores, configuration, travel lanes, auxiliary lanes, ramp radii, acceleration and deceleration lanes, taper lengths, freeway ramp terminals, and C-D roadways.

   A drawing or series of drawings should be provided showing the traffic volumes for all through and turning movements, as well as data on C-D roadways, local service roads, and origin-destination (O-D) travel particularly for weaving movements. The base-year or open-to-traffic-year AADT should be identified for the mainline, crossroads, ramps, and intersections. The design-year AADT, morning and evening DHVs, and trucks percentages for each movement should be included.

b. Highway Capacity Analysis. A narrative of the assumptions used and reasons for changes in the software default values should be included. Results of operational analysis, in the form of service levels for each element of the Interstate-route access facility, and for multiple years and periods of the day, should be provided on a drawing at a scale of 1:2000 to 1:4000.

The summary results, typically in levels-of-service (LOS), should be provided for each element, e.g., weaving, basic freeway ramp merge and diverge, ramp proper, at-grade signalized or unsignalized ramp terminals (intersections), crossroad arterial and its intersections in the access influence area for existing (no-build) and proposed (build) conditions in the base year or open-to-traffic year, and in the design year for morning and evening peak periods.

Queue analysis should be provided as part of the traffic operational analysis for those points where significant queuing may be expected, such as at ramp junctions with the crossroad and at each major intersection on the crossroad adjacent to an at-grade ramp terminal.

All highway capacity and operations calculations must be included in an Appendix to the IJ. If the nature of the project entails a level of traffic operations analysis generating an inordinately large volume of output, the bulk of the hand calculations and printout of the HCS or other software tools may be provided in electronic format (on a compact disc) if desired, rather than on a hardcopy. However, at least 10% of the points checked for LOS must be in hardcopy format. In this situation, a variety of points should be selected for the sample to be printed in paper format, especially critical locations. In addition, a hardcopy of each analyzed weaving area must be included in the Appendix.

An adjacent interchange, or intersection adjacent to the core access point/interchange, which is found to have a LOS below D for any of its elements, must be clearly identified. The IJ must contain a discussion of the impact this will have, if any, on the new or revised interchange(s) and Interstate-route mainline. Potential mitigation measures to alleviate adverse impacts to the core access point/interchange must be described to at least a concept level. An alternative would be to describe the mitigation measures in the IJ transmittal letter to FHWA or in a separate correspondence with FHWA.

c. Crossroad Highway Capacity Analysis. An intersection at a ramp terminal or along a crossroad must be analyzed to determine if it could have a negative impact on Interstate-route operations. A crossroad must be capable of collecting and distributing traffic to and from the Interstate route.

Each stop-controlled or signalized intersection within 400 m of the ramp terminal must be analyzed for traffic operation. It may be necessary to analyze an intersection on the crossroad beyond 400 m. It may be beneficial to assess traffic operational conditions 600 m or 800 m beyond the ramp limits. The exact intersections to be analyzed along the crossroad will be determined jointly by FHWA and the Department.

If the analysis shows that an adjacent intersection will operate at LOS of E or F in the design year, a LOS analysis must be done to determine when the adjacent intersection becomes unacceptable, i.e., below LOS of D.

An intersection that is shown to have a LOS of E or F in the open-to-traffic year or 7 years beyond must be investigated to at least a concept level to determined what needs to be done to make it operate at LOS of D or better in the design year, e.g., add lanes. It will be necessary to determine whether the failure is the result of normal traffic growth or the result of the interchange access change. The Department and the responsible local public agency will determine who will be responsible for necessary intersection improvements outside of the interchange area (to adjacent intersections) and when they will be accomplished. The Department will notify FHWA of the action to be taken either in the IJ, the IJ transmittal letter, or by separate correspondence.

Each intersection which is shown to have a LOS of E or F between years 7 and 20 will be monitored for needed improvements. The IJ, the IJ transmittal letter, or separate correspondence must identify who will be responsible for this activity.

48-1.03(07) FHWA Approval

Approval is required from the FHWA Washington, D.C., Headquarters office (HQ) for each major type of new or revised access request listed below. Two copies of the Final IJ must be sent to the FHWA Indiana Division office for an action of a significant nature requiring coordination with HQ. Advance coordination with HQ may be necessary for a complex or controversial project. For this situation, the Department should coordinate directly with the Division office, specifically, the appropriate Transportation Engineer.

1. FHWA Approval by HQ. HQ approval is required for each type 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;

c. establishing a new partial interchange of any form; or

d. establishing a new freeway-to-non-freeway (service) interchange in a Transportation Management Area (TMA). A TMA is defined as an urbanized area with a current population of more that 200,000 as determined by the most recent decennial census, or as an area for which the TMA designation is requested by the governor and the MPO or affected local officials, and officially designated by the Administrators of the FHWA and the Federal Trade Administration.

2. FHWA Approval by Division Office. One copy of the Final IJ must be sent to the Division office for approval for each type of Interstate-System new or revised access as follows:

   a. establishing a new freeway-to-non-freeway interchange not located in a TMA;

   b. modification of an existing freeway-to-non-freeway interchange configuration;

   c. establishing locked-gate access; or

   d. removal from service of ramps or interchanges.

FHWA approval of an IJ is valid for 10 years from the date of the letter granting its final approval. If 10 years have expired before proceeding with construction of the new or revised access, it will be necessary to re-evaluate the IJ. This involves obtaining current traffic data for that time, projecting such data out to 20 years and determining if the originally-approved IJ will still provide acceptable levels of service for the new design year. It will be necessary to repeat the procedures outlined herein and produce a revised IJ for FHWA approval.

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 guidelines to consider when determining the need for an interchange.

1. Functional Classification. An interchange should be provided at each freeway-to-freeway crossing. On a fully-access-controlled facility, an interchange should be provided with each major highway, unless this is determined inappropriate for other reasons. An interchange to each other type of highway 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. Where interchanges are spaced farther apart, freeway operations are improved. Spacing of urban interchanges between interchange crossroads should not be less than 1.5 km. 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 an urban area, a spacing of less than 1.5 km may be developed with grade-separated ramps or with collector-distributor roads. In a rural area, interchanges should not be spaced less than 5 km apart on the Interstate System or 3 km on another system.

4. Access. An interchange may be required where access availability from other sources is limited, and the freeway is the only facility that can practically serve the area.

5. Operations. A grade-separated facility without ramps will require a driver desiring to turn onto the crossroad to use another location to make his or her desired move. This will often improve the operation 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 Versus Underpass Roadway. 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. The decision is often based on features such as topography or functional classification.

48-2.0 INTERCHANGE TYPE SELECTION

48-2.01 General Evaluation

Section 48-2.02 provides the interchange types which may be used at a given site. The Office of Environmental Services' Environmental Services Team determines the type of interchange for the site. Typically, the Team will evaluate several types for potential application. Each type should be evaluated considering the following:

1. compatibility with the surrounding highway system;
2. route continuity;
3. level of service for each interchange element (e.g., freeway/ramp junction, ramp proper);
4. operational characteristics (single versus double exits, weaving, signing);
5. road user impacts (travel distance and time, safety, convenience, comfort);

6. driver expectancy (e. g., exit or entrance to the right);
7. geometric design;
8. construction and maintenance costs;
9. potential for stage construction;
10. right-of-way impacts and availability;
11. environmental impacts; and
12. potential growth of surrounding area.

Other overall factors which influence the selection of an interchange type are as follows:

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

2. Urban or Rural Area. In a rural area where interchanges occur relatively infrequently, the design can be selected strictly on the basis of service demand and analyzed as a separate unit. In an urban area 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. Each interchange should provide for all movements, even where the anticipated turning-traffic volume is low. An omitted maneuver may be a point of confusion to a driver searching for the exit or entrance. Unanticipated future development may increase the demand for that maneuver.

Figure 48-2A provides guidance for the types of interchanges that are adaptable to a freeway based on the functional classification of the intersecting facility in a rural, suburban or urban environment. At other than a freeway-to-freeway intersection, the choice of interchange will likely be limited to a cloverleaf or a diamond or a variation thereof.

48-2.02 Interchange Type

Each interchange must be custom-designed to fit the individual site considerations. The final design may be a minor or major modification of one of the basic types or may be a combination of two or more basic types described below.

48-2.02(01) Diamond

The diamond is the simplest and perhaps the most common type of interchange. A one-way diagonal ramp is provided in each quadrant with two at-grade intersections provided at the minor road. If these two intersections can be properly designed, the diamond is usually the best choice of interchange where the intersecting road is not access controlled. Figure 48-2B illustrates a schematic of a typical diamond interchange. Its advantages and disadvantages include the following.

1. <u>Advantages</u>.

   a. An exit from the mainline is made before reaching the crossroad structure. This conforms to driver expectancy and therefore minimizes confusion.

   b. Traffic can enter and exit the mainline at relatively high speed. Adequate sight distance can usually be provided, and the operational maneuvers are normally uncomplicated.

   c. Relatively little right of way is required.

   d. Left-turning maneuvers require little extra travel distance.

   e. The diamond configuration easily allows modifications to provide greater ramp capacity, if needed in the future. A spread diamond interchange has the potential for conversion to a cloverleaf.

   f. Its common usage has resulted in a high degree of driver familiarity.

2. <u>Disadvantages</u>.

   a. There are potential operational problems with the two at-grade intersections at the minor road. Signalization may be needed if the crossroad carriers moderate to high traffic volume. While a single-lane ramp may adequately serve traffic from the roadway, it may have to be widened to 2 or 3 lanes or be channelized for storage near the crossroad, in order to provide the required capacity.

   b. There is greater potential than, for example, a full cloverleaf for wrong-way entry onto a ramp. A median should be provided on the crossroad to facilitate proper channelization. Additional signing to should be placed to minimize improper use of a ramp.

   c. Sufficient intersection sight distance should be provided at the minor road.

48-2.02(02) Single-Point

Figure 48-2C illustrates a single-point interchange. All legs of the interchange meet at a single point. The advantages and disadvantages of this type include the following.

1. Advantages.

   a. The right-turn movements are typically free-flow movements. The design of a free-flow right turn should include an additional lane on the cross street beginning at the right-turn lane for at least 60 m before being merged. A free-flow right turn from the exit ramp to an arterial crossroad is not desirable where the nearest intersection on the crossroad is within 150 m, because of weaving.

   b. It can significantly increase the interchange capacity. This arrangement can alleviate the operational problems of having two closely-spaced at-grade intersections on the minor road. It overcomes the left-turn-lane storage problem for a driver wishing to enter the freeway.

   c. It reduces cross-street delays.

   d. It only requires one signal instead of the two required at a diamond.

   e. It reduces right-of-way needs.

   f. It can be used in a rural area where use of adjacent right of way is not desired due to environmental or other constraints.

2. Disadvantages.

   a. Channelization design must be considered to minimize driver confusion and the likelihood of a wrong-way maneuver. To provide positive guidance, at a minimum, dashed lines of 0.6-m length should be placed through the intersection.

   b. There is a significantly wider pavement area for a pedestrian to cross a ramp. The design should provide for a pedestrian to cross the minor roadway at an adjacent intersection, instead of the ramp terminal intersection.

   c. Because of wide pavement areas, it requires longer signal clearance intervals.

   d. It is difficult to accommodate a frontage road.

e. It has a higher construction cost than a diamond because of the need for a larger structure. However, this is often offset by the reduced right-of-way cost.

f. The design process becomes more difficult if the skew angle of the interchanging roadways approaches 30 deg.

g. It is difficult to add capacity in the future.

48-2.02(03) Three-Level Diamond

Figure 48-2D illustrates a three-level diamond. All of the at-grade intersections are on a separate level than the two mainlines. Advantages and disadvantages include the following.

1. Advantages.

    a. It can handle high traffic volume.

    b. At-grade intersections are removed from both mainlines, thereby significantly increasing the capacity of the intersection.

    c. It requires less right-of-way than loop ramps.

    d. A one-way frontage road can be easily incorporated into the interchange configuration.

2. Disadvantages.

    a. To make a left turn, a driver needs to pass through three at-grade intersections or traffic signals.

    b. The additional structures result in higher construction costs.

48-2.02(04) Full-Cloverleaf

A cloverleaf interchange is used at a 4-leg intersection and employs loop ramps to accommodate left-turn movements. A loop may be provided in any quadrant. A full-cloverleaf interchange is that with a loop in each quadrant. A partial-cloverleaf interchange is that with a loop missing from at least one quadrant.

Where two access-controlled highways intersect, a full cloverleaf is the minimum type of interchange design that will suffice. However, a cloverleaf introduces undesirable operational features such as double exit or entrance from the mainline, weaving between entering and exiting vehicles with the mainline traffic and, if compared to a directional interchange, the additional travel time and distance for a left-turning vehicle. Therefore, a collector-distributor (C-D) road should be considered with a full cloverleaf, or a fully-directional interchange should be provided. Figure 48-2E provides examples of a full cloverleaf with or without C-D roads. See Section 48-6.03 for a discussion on C-D roads.

Operational experience with a full-cloverleaf interchange has yielded conclusions regarding its design. Subject to a detailed analysis, the following characterize the design of a cloverleaf.

1. Design-Speed Impacts. For an increase in design speed, there will be an increase in the following:

   a. travel distance;
   b. required right of way; and
   c. travel time.

2. Loop Radius. A loop can be practically designed for an approximate radius of 55 to 75 m. A smaller radius is used in urban area, while a larger radius is used in a rural area.

3. Loop Geometry. A circular-curve loop ramp is the most desirable geometrically because speed and travel path tend to be more constant and uniform.

4. Loop Capacity. Expected design capacity for a single-lane loop ranges from 800 to 1200 vph. For a 2-lane loop, this is 1000 to 2000 vph. The higher figures are only achievable where the design speed is 50 km/h or higher and few trucks use the loop.

5. Weaving-Area Volume. An auxiliary lane is provided between successive entrance or exit loops within the interior of a cloverleaf interchange. This produces a weaving area between the mainline and entering or exiting traffic. Where the total volume on the two successive ramps reaches approximately 1000 vph, interference increases rapidly with a resulting reduction of the through-traffic speed. At this weaving-volume level, a collector-distributor road should be considered.

6. Weaving-Area Length. The minimum weaving-area length between the exit and entrance gores of loops on a new cloverleaf interchange without C-D roads or that are undergoing major reconstruction should be at least 300 m or the distance determined by from a capacity analysis, whichever is greater.

7. Advantages and Disadvantages. These include the following.

a. Advantages.

(1) A full cloverleaf is intended to eliminate all vehicular stops through the use of merges.

(2) A full cloverleaf eliminates at-grade intersections and, therefore, eliminates left turns.

(3) Where right of way is reasonably inexpensive and adverse impacts are minimal, a full cloverleaf is a practical option.

b. Disadvantages.

(1) A full cloverleaf requires more right-of-way and is more costly than a diamond.

(2) A loop results in a greater travel distance for a left-turning vehicle than does a diamond, and the vehicle operates at a lower speed.

(3) At least one exit or entrance is located beyond the crossroad structure, which does not conform to driver expectancy.

(4) A full cloverleaf may introduce signing problems.

(5) A full cloverleaf results in weaving areas. If the sum of traffic counts on two adjoining loops approaches 1,000 vehicles per hour, interference mounts rapidly, resulting in a reduction of speed of through traffic. Consideration should be given to adding a collector-distributor road. The use of acceleration or deceleration lanes is an alternative to a collector-distributor road.

(6) A ramp at a diamond interchange can be easily widened to increase capacity; while a two-lane loop ramp requires at least two additional lanes (one on each side) through the separation structure, longer weaving-area distance, and a larger loop radius to operate.

(7) A pedestrian movement along a cross street is difficult to safely accommodate.

(8) 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.

48-2.02(05) Partial-Cloverleaf

A partial-cloverleaf interchange is that with a loop in each of one, two, or three quadrants. It is appropriate where right-of-way restrictions preclude a ramp in one or more quadrants. It is also advantageous where a left-turn movement can be provided onto the major road from a loop without the immediate presence of an entrance loop from the minor road. Figure 48-2F illustrates examples of a partial cloverleaf. In details B and C, both left-turn movements onto the major road are provided from loops, a distinct preference.

An interchange ramp in only one quadrant has 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-traffic volume does not justify the separation, a single two-way divided ramp of near minimum design 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 a turning movement to be made with a right-turn exit or entrance.

The advantages and disadvantages listed for a full-cloverleaf also apply to a partial-cloverleaf (e.g., geometric restriction of loop). The specific advantages of a partial cloverleaf include the following.

1. Depending upon site conditions, a partial-cloverleaf may offer the opportunity to increase weaving-area distance.

2. A partial-cloverleaf is appropriate where one or more quadrants present adverse right-of-way or terrain problems.

3. A partial-cloverleaf may reduce the number of left-turn movements when compared to a diamond interchange.

4. A partial-cloverleaf design with loops in opposite quadrants is desirable because it eliminates the weaving problem associated with a full-cloverleaf design.

48-2.02(06) Three-Leg

A three-leg interchange, also known as a T or Y interchange, is provided at an intersection with three legs. Figure 48-2G illustrates examples of 3-leg interchanges with methods of providing the turning movements. See the AASHTO *Policy on Geometric Design of Highways and Streets* for additional variations of the three-leg interchange. The trumpet type is shown in detail A where three of the turning movements are accommodated with direct or semi-direct ramps and one movement by a loop ramp. The semi-direct ramp should favor the higher-traffic-volume left-turn movement and the loop the lighter volume. Where both left-turning movements are fairly common, the design in detail B is more suitable. A fully-directional interchange (detail C) is appropriate where all turning-traffic volumes are high, or the intersection is between two access-controlled highways. This would be the most costly type because of the necessary multiple structures. A three-leg interchange should only be considered where future expansion in the unused quadrant is either impossible or highly unlikely. It is difficult to expand or modify in the future.

48-2.02(07) Directional or Semi-Directional

The following definitions apply to a directional or semi-directional interchange.

1. Directional Ramp. A ramp that does not deviate from the intended direction of travel (see Figure 48-2H).

2. Semi-Directional Ramp. A ramp that is indirect in alignment, yet more direct than a loop (see Figure 48-2 I).

3. Fully-Directional Interchange. An interchange where the left-turn movement is provided by a directional ramp (see Figure 48-2H).

4. Semi-Directional Interchange. An interchange where the left-turn movement is provided by a semi-directional ramp, even if the minor left-turn movement is accommodated by a loop (see Figure 48-2 I).

A directional or semi-directional ramp is used for a high-traffic-volume left-turn movement to reduce travel distance, to increase speed and capacity, and to eliminate weaving. This type of connection allows an interchange to operate at a better level of service than is possible with a cloverleaf interchange. A left-hand exit or entrance may violate driver expectancy and, therefore, should be avoided.

A directional or semi-directional interchange is warranted in an urban area at a freeway-to-freeway or freeway-to-arterial intersection. It requires less right-of-way than a cloverleaf. 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. An interchange involving two freeways will almost always require directional layouts.

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 a significant distance. For example, a lane should not be dropped at the exit of a diamond interchange and then added at the downstream entrance because the traffic volume between the exit and entrance drops significantly. Likewise, a basic lane should not be dropped between closely-spaced interchanges 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 principles which apply at a freeway exit or entrance as follows.

1. Exit. At an exit, 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 a cloverleaf-loop-ramp exit which follows a loop ramp entrance or at an exit 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 450 m and a continuous auxiliary lane between the terminals is being used). 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.

2. Entrance. At an entrance, 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.

For example, dropping two lanes at a 2-lane exit ramp would violate the principle of lane balance. One lane should provide the option of remaining on the freeway. Lane balance would also prohibit immediately merging both lanes of a 2-lane entrance ramp into a highway mainline without the addition of at least one additional lane beyond the entrance ramp. Figure 48-3B illustrates how to coordinate lane balance and the basic number of lanes at an interchange. Figure 48-3A illustrates how to achieve lane balance at the merging and diverging points of branch connections.

48-3.03 Route-Number Continuity

Each highway with an interchange is designated with a route number. A through-traveling driver should be provided a continuous numbered route on which changing lanes is not necessary to continue on the through route. Route-number continuity is consistent with driver expectancy, simplifies signing, and reduces the decision demands on the driver. An interchange configuration should not favor the higher-traffic-volume movement, but rather, the through-route's number.

48-3.04 Signing and Marking

Proper interchange operation depends 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 Highway Operations Division's Office of Traffic Engineering will determine the use of traffic-control devices at an interchange.

48-3.05 Uniformity

Each interchange along a freeway should be reasonably uniform in geometric layout and appearance. Except for a highly-specialized situation, each entrance or exit ramp should be to the right.

48-3.06 Distance Between Successive Freeway-Ramp Junctions

In an urban area, successive freeway-ramp junctions frequently may need to be placed relatively close to each other. The distance between the junctions should provide for vehicular maneuvering, signing, and capacity. The ramp-pair combinations are entrance followed by entrance (EN-EN), exit followed by exit (EX-EX), exit followed by entrance (EX-EN), and entrance followed by exit (EN-EX). The final decision on the spacing between freeway-ramp junctions will be based on the level-of-service criteria and on the capacity methodology described in the *Highway Capacity Manual*.

48-3.07 Auxiliary Lane

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

1. Within Interchange. Figure 48-3D provides the schematics of alternative designs for adding and dropping an auxiliary lane within interchanges. The selected design will depend upon traffic volume for the exiting, entering, and through movements.

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

Details for exits and entrances are provided in Section 48-4.0, and details for a lane drop are provided in Section 48-6.02.

48-3.08 Lane Reduction

A reduction in the basic number of lanes 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 traffic 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.

If a lane reduction of a basic lane or an auxiliary lane is made within an interchange, it should be made in conjunction with a two-lane exit, or in a single-lane exit with an adequate recovery lane. If a basic lane or auxiliary lane is to be dropped between interchanges, it should be accomplished at a distance of 600 to 900 m from the previous interchange to allow for adequate signing.

The lane reduction should be made on the driver's right side following an exit ramp, since there is likely to be less traffic in that lane. The end of the lane drop should be tapered into the highway in a manner similar to that at a ramp entrance. The rate of taper should be longer than that for a ramp. The desirable taper rate should be 70:1, with a minimum rate of 50:1.

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. The following summarizes major safety considerations.

1. Exit Point. An existing interchange may have been built with an exit point which cannot clearly be seen by an approaching driver. Decision sight distance should be provided where practical at a freeway exit. The pavement surface should be used for the height of object (0

m.). A 150-mm height of object is acceptable. See Section 48-4.01 for the application of decision sight distance to a freeway exit. Proper advance signing of the exit is essential.

2. Exit-Speed Change. A freeway exit 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. Merge. A rear-end collision in an entrance merge 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. An interchange can be a significant source of driver confusion. Therefore, it should be designed to conform to the principles of driver expectation. A left-hand merge is 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, a left-hand exit or entrance should not be used, because it is not consistent with driver expectancy when it is mixed with a right-hand entrance or exit. An exit should not be placed in line with the freeway tangent section at the point of mainline curvature to the left.

5. Fixed Object. Because of traffic operations at an interchange, a fixed object may be located within an interchange, such as a sign at an exit gore or a bridge pier or railing. It should be removed where practical, made breakaway, or shielded with a barrier or crash cushion. Horizontal stopping sight distance should be considered. With the minimum radius for a given design speed, the normal lateral clearance at an underpass pier or abutment does not provide the minimum stopping sight distance. Thus, an above-minimum radius should be used for horizontal curvature on a highway through an interchange. See Chapter Forty-nine.

6. Wrong-Way Entrance. A wrong-way driving maneuver originates at an interchange. It sometimes cannot be avoided, but it may result from driver confusion due to poor visibility, confusing ramp arrangement, or inadequate signing. The interchange design must attempt to minimize the possibility of a wrong-way entrance.

7. Weaving. An area of vehicular weaving may create a high demand on driver skills and attentiveness. Where practical, an interchange 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 an interchange is not present,
2. freeway-ramp junction,
3. weaving area,
4. ramp proper, and
5. ramp and crossroad intersection.

The capacity reference is the *Highway Capacity Manual* (HCM). The HCM provides the analytical tools to analyze the level of service for each element listed above.

The interchange should operate at an acceptable level of service. The values shown in Figures 53-1 and 54-2A for a freeway will also apply to an interchange. The level of service for each interchange element should be the same as the level of service provided on the basic freeway section. Interchange elements should be more than one level of service below that of the basic freeway section. The operation of the ramp-crossroad intersection in an urban area should not impair the operation of the mainline. This will involve a consideration of the operational characteristics on the minor road for some distance in either direction from the interchange. For a State-route project, the Office of Environmental Services' Environmental Policy Team is responsible for conducting the preliminary capacity analyses at an interchange.

48-3.11 Testing for Ease of Operation

The designer should review the proposed design from the driver's perspective. This involves tracing each possible movement 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 distance and sight distance is available. The designer should know the peak-hour traffic volume, number of traffic lanes, etc., so as to determine the type of traffic the driver will encounter.

48-4.0 FREEWAY-RAMP JUNCTION

48-4.01 Exit Ramp

48-4.01(01) Types

There are two types of exit junctions, the parallel design and the taper design. Figure 48-4A illustrates these. For a new or reconstructed ramp, the parallel design shown in illustration A should be used. An existing taper design as shown in illustration B may be retained if deemed acceptable and there is not an adverse history of accidents at the ramp junction. However, the designer may want to consider replacing an existing taper design with a parallel design as follows:

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 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 result in smoother operation;

4. the need is satisfied for a continuous auxiliary lane (see Section 48-3.07); and

5. the capacity of the at-grade ramp terminal is insufficient and ramp traffic may back up onto the freeway.

The INDOT *Standard Drawings* provide detail information for a parallel exit-ramp junction. For design information on a taper-ramp junction, see AASHTO's *A Policy on Geometric Design of Highways and Streets*.

48-4.01(02) Taper Length

For a parallel-lane exit, the taper rate applies to the beginning taper of the parallel lane. This distance is 30 m as illustrated in Figure 48-4A.

48-4.01(03) Deceleration

Sufficient deceleration distance is needed to safely and comfortably allow an exiting vehicle to leave the freeway mainline. Deceleration should occur within the full width of the parallel exit lane. The 300-m length of deceleration shown in Figure 48-4A and the INDOT *Standard Drawings* will accommodate each design speed or grade. It should always be used unless restricted conditions are present such as topographical features, adverse impacts, existing geometry, etc., which will not permit the use of the typical deceleration configuration.

48-4.01(04) Sight Distance

Decision sight distance should be provided for a driver approaching a freeway exit. This sight distance is particularly important for an exit loop immediately beyond a structure. Vertical curvature or a bridge pier can obstruct the exit point. For determining adequate sight distance, the height of object will be 0 mm (the roadway surface). However, it is acceptable to use 150 mm. Chapter Forty-two discusses decision sight distance in more detail.

48-4.01(05) Superelevation

Superelevation for a horizontal curve in the vicinity of the ramp junction must be developed to properly transition the driver from the mainline to the curvature at the exit. The principles of superelevation for an open highway, as discussed in Chapter Forty-three, should be applied to the 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 is 70 km/h or lower. The following will apply to superelevation development at an exit ramp.

1. e_{max}. On the exit ramp portion of the ramp junction, e_{max} is 8%.

2. Superelevation Rate. As discussed in Section 43-3.0, Method 5 is used for an open highway to distribute superelevation and side friction. Therefore, Figure 43-3A(1) will be used to determine the proper superelevation rate. The designer should use the ramp design speed and the curve radius to read into the table to determine e, subject to R_{min} for the ramp design speed. The superelevation rate and radius used should reflect a decreasing sequence of design speed for the exit terminal, ramp proper, and at-grade terminal for a diamond-interchange ramp.

3. Transition Length. The designer should use the superelevation transition length for a 2-lane roadway as shown in Figure 43-3A(1) to transition the exit-ramp cross slope to the superelevation rate at the PC.

4. Distribution. The superelevation-transition length should be distributed such that 60 to 80% of the length is in advance of the PC and the remainder beyond the PC. However, at a ramp junction, 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. The axis of rotation is about the centerline of the ramp travelway.

48-4.01(06) Cross-Slope Rollover

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

1. To Physical Nose. The cross-slope rollover should not exceed the ranges as follows:

Design Speed, km/h	Rollover, %
> 60	4 to 5
40 or 50	5 to 6
≤ 30	5 to 8

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

3. Drainage Inlet. Where required, this is placed between the physical gore and gore nose. The presence of a drainage inlet may require two breaks in the gore cross slope. The breaks should be in accordance with Item 1 or 2 above, depending on the inlet location.

See Section 48-4.01(08) for nose definition.

48-4.01(07) Shoulder Transition

The right shoulder of the mainline will be transitioned to the narrower shoulder of the ramp. As illustrated in Figure 48-4A and the INDOT *Standard Drawings*, the shoulder width along the mainline will be maintained until 30 m before the gore nose or ramp PC. The shoulder width will then be transitioned to the ramp right shoulder width, typically 2.4 m. In a restricted area, it is acceptable to provide a 1.8-m minimum right shoulder width along the entire parallel exit ramp area.

48-4.01(08) Gore Area

The term gore indicates an area downstream from the intersection point of the mainline and exit shoulders. The gore area is considered to be both the paved triangular area between the through lane and the exit ramp, plus the graded area which may extend 100 m downstream beyond the gore nose. The following definitions will apply (see Figure 48-4B).

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

2. Dimension Nose. This is the point where the shoulder is considered to begin within the gore area. For an exit ramp, the dimension nose is 1.2 m wide.

3. Physical Nose. This is the point where the ramp and mainline shoulders converge. As illustrated in Figure 48-4B, the physical nose has a width of 4.2 m.

4. <u>Gore Nose</u>. This is the point where the paved shoulder ends and the sodded area begins as the ramp and mainline diverge from one another. As illustrated in Figure 48-4B, the gore nose has a width of 1.8 m and does not include the shoulders.

The following should be considered when designing the gore.

1. <u>Obstacle</u>. If practical, the area beyond the gore nose should be free of obstacles (except the ramp exit sign) for at least 30 m beyond the gore nose. An obstacle within 100 m of the gore nose is to be made breakaway or shielded by a barrier. See Section 49-3.0.

2. <u>Side Slope</u>. 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. This area will be non-traversable. The gore must shield the motorist from this area. The vertical divergence of the ramp and mainline may warrant protection for both roadways beyond the gore (see Section 49-3.0).

3. <u>Cross Slope</u>. The paved triangular gore area between the through lane and exit ramp should be safely traversable. The cross slope is the same as that of the mainline (typically 2%) from the painted nose to the dimension nose. Beyond this point, the gore area is depressed with a cross slope of 2 to 4%. See Section 48-4.01(06) for criteria on breaks in cross slope within the gore area.

4. <u>Traffic-Control Devices</u>. Signing in advance of the exit and at the divergence should be in accordance with the MUTCD and Chapter Seventy-five. See Chapter Seventy-six for pavement-marking details for the triangular area upstream from the gore nose.

<u>48-4.02 Entrance Ramp</u>

48-4.02(01) Types

There are two types of entrance-ramp junctions, the parallel design and the taper design. Figure 48-4C illustrates these. The parallel design should be used for a new or reconstructed ramp as shown in Illustration A. The parallel design offers advantages over the taper design as follows.

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

2. Where the acceleration length needs to be lengthened for a grade or truck, the parallel design provides a longer distance.

3. Where there is insufficient sight distance available for the driver to merge into the mainline (e.g., where there is a sharp curve 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 incorporated into the design of the continuous auxiliary lane.

The INDOT *Standard Drawings* provide details for a parallel entrance-ramp junction. For design information on a taper entrance, see AASHTO's *A Policy on Geometric Design of Highways and Streets*.

48-4.02(02) Taper Length

The taper length at the merge point is a minimum of 90 m as illustrated in Figure 48-4C.

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 depend on the design speed of the last controlling horizontal curve on the entrance ramp and the design speed of the mainline. If determining the acceleration length, the designer should consider the following:

1. <u>Passenger Car</u>. Figure 48-4D provides the minimum length of acceleration for a passenger car. The acceleration distance is measured from the PT of the last controlling curve to the beginning of the taper (see Figure 48-4C). Where an upgrade is steeper than 2% over the acceleration distance, the acceleration length should be adjusted according to the value shown in Figure 48-4E.

   The acceleration length provides sufficient distance for acceleration of a passenger car. Where the mainline and ramp will carry traffic volume approaching the design capacity of the merging area, the available acceleration distance should total 375 m, exclusive of the taper, to provide additional merging opportunity. This distance is measured from the PT of the ramp entrance curve.

2. <u>Truck</u>. Where there are a significant number of trucks to govern the design of the ramp, the truck-acceleration distance provided in Figure 48-4F should be considered. Trucks may govern the ramp design at a weigh station, truck stop, or transport staging terminal. At

another type of ramp entrance, the truck-acceleration distance should be considered where there is substantial entering truck traffic and as follows:

a. there is LOS of D or lower at the junction;

b. there is a significant accident history involving trucks which can be attributed to an inadequate acceleration length; or

c. there is an undesirable level of vehicular delay at the junction attributed to an inadequate acceleration length.

Where an upgrade is steeper than 2%, the truck-acceleration distance may be corrected for grade. Figures 44-2B and 44-2C provide performance criteria for a truck on an accelerating grade. Before providing additional acceleration lane length, the designer must consider the impacts of the added length (e.g., additional construction costs, wider structure, and right-of-way impacts).

3. Horizontal Curve. The specific application of the acceleration criteria to a horizontal curve is as follows:

a. The design speed of the last horizontal curve on the ramp proper will be determined from the open-highway condition. This is discussed in Section 43-2.0.

b. For a relatively short entrance ramp, the acceleration distance may be determined as the 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 be provided for a driver on the mainline approaching an entrance terminal. He or she needs sufficient distance to see the merging traffic so he or she can adjust speed or change lanes to allow the merging traffic to enter the freeway. Likewise, a driver on an entrance ramp needs 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 an open highway, as discussed in Section 43-

3.01, should be applied to the entrance-ramp design. Section 48-4.01 provides the superelevation criteria for an exit-ramp junction which is also applicable to an entrance-ramp junction. This includes e_{max}, superelevation rate, transition length, the distribution of transition length 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 the two are adjacent to each other. The maximum algebraic difference is 4% to 5% beyond the physical nose. Between the gore nose and physical nose, the maximum cross slope rollover is 8%. See Section 48-4.02(08) for gore-area definition.

48-4.02(07) Shoulder Transition

At an entrance terminal, the right shoulder must be transitioned from the narrower ramp shoulder to the wider freeway shoulder. Figure 48-4C and the INDOT *Standard Drawings* illustrate this shoulder transition. In a restricted area, it is acceptable to maintain the1.8-m right shoulder width on the ramp throughout the parallel lane until the merge with the mainline.

48-4.02(08) Gore Area

Section 48-4.01(08) provides the definitions for the nose type which may be within the gore area. The following provides the nose dimensions for an entrance gore.

1. Painted Nose. The painted-nose dimension is considered to be 0 m (i.e., the point where the two paint lines meet).

2. Dimension Nose. The dimension-nose width is 0.6 m.

3. Physical Nose. The physical nose has a width of 4.2 m.

4. Gore Nose. The gore nose has a width of 1.8 m.

48-4.03 Continuous Auxiliary Lane

For closely-spaced interchanges, it may be warranted to provide a continuous auxiliary lane between the entrance ramp of one interchange and the exit ramp of the downstream interchange. A continuous auxiliary lane should be considered as follows:

1. the distance between the end of the entrance taper (without the connecting auxiliary lane) and the beginning of the downstream exit taper is 450 m or less; or

2. a capacity or operational analysis indicates the need.

48-4.04 Multi-Lane Terminal

A multi-lane terminal may be required if the capacity of the ramp is too great for single-lane operation. It may also be used to improve traffic operations (e.g., weaving) at the junction. The following lists considerations if multi-lane terminal is required.

1. Lane Balance. Lane balance at the ramp junction should be maintained. See Section 48-3.02.

2. Loop Ramp. Where the capacity analysis indicates that single-lane loop capacity is insufficient, consideration should be given to providing either a 2-loop ramp or a direct-connection ramp. For a 2-lane loop ramp, the designer should consider the following.

   a. A two-lane loop ramp with a short radius is not recommended because a driver is adverse to driving side-by-side with another vehicle and, therefore, tends to drive the ramp as a single-lane loop.

   b. Expected design capacity for a single-lane loop ranges from 800 to 1200 vph, and for a 2-lane loop, 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. See Items 3 and 4 below.

3. Entrance. For a multi-lane entrance ramp, a parallel-lane design should be used. Figure 48-4G illustrates a schematic of a multi-lane entrance ramp.

4. Exit. For a 2-lane exit ramp, the additional lane should be added at least 400 m prior to the terminal. The total length from the beginning of the first taper to the gore nose will range from 760 m for turning-traffic volume of 1500 vph or less, to 1070 m for turning-traffic volume of 3000 vph. Figure 48-4H illustrates a schematic of a parallel-lane multi-lane exit ramp.

   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 760-m length indicated above. The exit taper to the parallel deceleration lanes should be 60 m in length. This parallel deceleration lane concept should be considered where vehicle storage is anticipated in the ramp lanes and deceleration lanes in advance of the crossroad intersection.

5. Signing. The geometric layout of must be coordinated with the Office of Traffic Engineering because of the signing which may be required in advance of the exit.

48-4.05 Fork or Branch Connection

Figures 48-4 I and 48-4J illustrate details for a fork or branch connection. The geometric considerations are as follows.

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 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 48-4 I, Schematic A, B, or C. Where one of the roadways is on a tangent at a fork, the gore design should be the same as for a multi-lane exit ramp. See Figure 48-4 I, Schematic D.

3. Nose Width. At the painted nose of a fork, the lane should be at least 7.2 m wide but not over 8.6 m. The widening from 3.6 m to 7.2 m should occur within a distance of 300 m to 550 m. See Figure 48-4 I, Schematic A.

   If a design-hourly volume of greater than 1500 is anticipated on the exit ramp at a fork on a systems interchange, the exit deceleration lanes, exclusive of the exit tapers, should begin approximately 1600 m ahead of the painted gore nose.

4. Branch Connection. Where traffic is expected to merge, a full lane width should be carried for at least 300 m beyond the painted nose. See Figure 48-4J, Schematic B.

48-5.0 RAMP DESIGN

48-5.01 Design Speed

Figure 48-5A provides the acceptable ranges of ramp design speed based on the design speed of the mainline. The highway with the greater design speed should control in selecting the design speed for the ramp. However, the ramp design speed may vary. The portion of the ramp closer to the lower-speed highway should be designed for a lower speed. The designer should consider the following:

1. Freeway-Ramp Junction. The design speed shown in Figure 48-5A applies to the ramp proper and not to the ramp junction. The ramp junction is designed using the freeway mainline design speed.

2. At-Grade Terminal. If a ramp will be terminated at an at-grade intersection with a stop or signal control, the design speed shown in Figure 48-5A may not be applicable to the ramp portion near the intersection. For additional information on design-speed selection near an at-grade intersection, see Chapter Forty-six.

3. Variable Speed. The ramp design speed may vary based on the two design speeds of the intersecting roadways. A higher design speed should be used on the portion of the ramp near the higher-speed facility while a lower design speed may be selected near the lower-speed facility. If using a variable design speed, the maximum speed differential between controlling design elements (e.g., horizontal curve, reverse curve) should not be greater than 20 to 30 km/h. Sufficient deceleration distance should be made available between design elements with varying design speeds (e.g., two horizontal curves).

4. Ramp for Right Turn. The design speed for a right-turn ramps is in the mid- to high range. This includes, for example, a diagonal ramp of a diamond interchange.

5. Loop Ramp. A design speed in the high range is not attainable for a loop ramp. The minimum value controls. For a mainline design speed of 80 km/h or higher, the loop design speed should not be lower than 40 km/h. However, a design speed higher than 50 km/h will require more right-of-way and may not be practical in an urban area. A loop design speed should not be higher than 60 km/h. An arterial loop ramp radius should be greater than 45 m.

6. Semidirect Connection. A design speed in the mid- to high range should be used for a semidirect connection. A design speed lower than 50 km/h should not be used. A design speed of 80 km/h or higher is not practical for a short, single-lane ramp. For a 2-lane ramp, a design speed in the mid- to high range should be used.

7. Direct Connection. For a direct connection, the design speed should be in the mid to high range. The design speed should be at least 70 km/h but, as a minimum, should not be lower than 60 km/h.

48-5.02 Cross Section

The INDOT *Standard Drawings* provide typical cross sections for tangent or superelevated ramps. The following will also apply to the ramp cross section.

1. Width. The minimum paved width of a 1-way, 1-lane ramp will be 8.5 m. This width includes a 1.2-m left shoulder, a 2.4-m right shoulder, and a 4.9-m travelway. A multi-lane ramp travelway width should be in a multiple of 3.6 m, with a 1.2-m wide left shoulder and a 3.0-m wide right shoulder. The guardrail offset from the edge of shoulder should be 0.6 m. The bridge railing offset should be 0.5 m. Full-depth paving equal to the ramp-pavement thickness should be provided on the shoulders because of frequent use of shoulders for a turning movement or passing a stalled vehicle.

2. Pavement Design. A loop ramp or other ramp with a horizontal-curve radius less than or equal to 100 m should be designed with full-depth pavement for the entire width. For a ramp with a horizontal-curve radius of greater than 100 m, only the 4.9-m traveled way will have a full-depth pavement structure. An outer connector ramp at a cloverleaf-type interchange, or a ramp at a diamond-type interchange, should have full-depth shoulders. For additional pavement design information, see Chapter Fifty-two and the ramp cross-section figures in Section 45-8.0.

3. Cross Slope. The traveled-way cross slope is 2%. The right-shoulder cross slope is 4%. The left-shoulder cross slope is 2% and slopes away from the traveled way. For a superelevated ramp, the entire ramp width should have the same cross slope.

4. Curbs. Curbs should not be used on a ramp. However, an asphalt sloping curb may used for drainage or to prevent erosion on a steep embankment slope. See Section 49-3.04 for additional curb information.

5. Bridge or Underpass. The full paved width of the ramp should be carried over a bridge or beneath an underpass. The clear width below an underpass should also include that of the clear zone.

6. Side Slope or Ditch. A side slope or ditch should be in accordance with the same criteria as for the mainline. Section 45-3.0 and Section 45-8.0 provide additional information on the design of these elements.

7. Clear Zone. The clear zone from the edge of the traveled way portion of the ramp will be determined from Figure 49-2A. The design AADT will be the directional AADT for the ramp.

8. Barrier. An additional 0.6 m should be added to the shoulder width where a roadside barrier is required. Where a barrier is present on a horizontal curve, the designer should determine the barrier impact on horizontal sight distance. See Section 43-4.04.

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

48-5.03 Horizontal Alignment

48-5.03(01) Theoretical Basis

Establishing horizontal alignment criteria for a 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-Road Condition. Chapter Forty-three discusses the theoretical basis for horizontal alignment assuming the open-road condition. In summary, this includes the following:

    a. relatively low side-friction factor (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 grades for superelevation-transition length; and

    d. distributing 50% to 70% of the superelevation-transition length to the tangent and the remainder to the horizontal curve.

2. Turning-Roadway Condition. Section 46-3.02 discusses the theoretical basis for horizontal alignment assuming the turning-roadway condition. In summary, this includes the following:

    a. higher side-friction factor than the open-road condition to reflect a higher level of driver acceptance of discomfort;

    b. a range of acceptable superelevation rates for a combination of curve radius and design speed to reflect the need for flexibility to meet field conditions for turning-roadway design; and

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

For an interchange ramp, the selection of which theoretical basis to use will be based on the portion of the ramp under design as follows:

1. freeway-ramp junction;

2. ramp proper (directional ramp);
3. ramp proper (loop ramp);
4. ramp terminus (intersection control); or
5. ramp terminus (merge control).

The general controls that dictate horizontal alignment on an interchange ramp are discussed below.

48-5.03(02) General Controls

The following will apply to the horizontal alignment.

1. Superelevation Rate (Rural). For a non-loop ramp in a rural areas, the superelevation rate will be based on e_{max} = 8% and the open-road condition. See Figure 48-5B for the specific superelevation rate based on ramp design speed and curve radius.

2. Superelevation Rate (Urban). For a ramp in an urban area, the superelevation rate will be based on e_{max} of 4%, 6%, or 8%, depending on site conditions. The highest practical rate should be used, especially for a descending ramp. The open-road condition will be used, as it is acceptable to assume the turning-roadway condition. Figure 48-5C provides the specific superelevation rate for e_{max} = 6%. Figure 48-5D provides that for e_{max} = 4% using the open-road condition. For the turning-roadway condition, see Section 46-3.02.

3. Superelevation Transition. The open-road condition, as discussed in Section 43-3.0, will also apply for transitioning to and from the needed superelevation on a ramp. This includes the relative longitudinal grades shown in Figure 43-3E, which have been used to calculate the superelevation runoff length shown in Figures 48-5B, 48-5C and 48-5D. The methodology described in Section 43-3.0 is used to calculate the superelevation runoff and tangent runout distance with the following modifications.

   a. One-Lane Ramp. The width of rotation, *W*, is assumed to be one-half the travelway width (0.5 x 4.9 = 2.45 m). With this assumption, the minimum length shown in Figure 48-5B, 48-5C, or 48-5D, column A, applies to a one-lane ramp.

   b. Two-Lane Ramps. The width of rotation, *W*, is assumed to be one-half of the widest travelway, which is determined from the minimum radius R = 55 m for the lowest ramp design speed of 40 km/h (0.5 x 8.2 = 4.1 m).

4. Minimum Length of Design Superelevation. The designer should not superelevate a curve on ramp such that the design superelevation rate is maintained on the curve for a very short distance. The minimum distance for design superelevation should be about 30 m.

5. Axis of Rotation. This will be about the centerline of the ramp travelway.

6. Shoulder Superelevation. The criteria described in Section 43-3.0 for superelevating the high-side or low-side shoulder on an open roadway will apply to a superelevated curve on a ramp. The entire ramp width should have the same cross slope.

7. Reverse Curve. To meet restrictive right-of-way requirements, a ramp may be designed with a reverse curve. The reverse curve should be designed with a tangent section between them. For a ramp, however, it is necessary to provide a continuously rotating plane between the two curves. If a continuously rotating plane is used, the distance between the PT and the succeeding PC should be 30 m. It is acceptable for the PT and PC to be coincident. See Section 43-3.0 for more information on superelevation at a reverse curve.

8. Sight Distance. Section 43-4.0 provides the criteria for sight distance around a horizontal curve based on the curve radius and design speed. The criteria also apply to a curve on a ramp. 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 Junction

Horizontal alignment at a freeway-ramp junction is based on the open-road condition. This is discussed in Section 48-4.0.

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

A directional ramp is a ramp which is relatively direct in its alignment. This includes a ramp at a diamond interchange, an outer ramp at a cloverleaf interchange, or a ramp at a directional or semi-directional interchange.

The ramp proper, for the purpose of horizontal alignment, is considered to begin at the gore nose of an exit ramp, or to end approximately 45 m before the dimension nose of an entrance ramp. See the discussion in Section 48-5.03(01) to determine where the open-road condition or turning-roadway condition applies to the horizontal alignment on a directional ramp.

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

A loop ramp is that on the interior portion of a cloverleaf or partial-cloverleaf interchange. The ramp proper is considered to begin at approximately the physical nose of an exit ramp, or to end at approximately the physical nose of an entrance ramp. Because of the normally restrictive conditions for a loop ramp, the curve radius is less than 100 m. Therefore, it is desirable to use the open-road condition for horizontal alignment; although it is more practical to use the turning-roadway condition.

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

An interchange ramp may end at an at-grade intersection. This may be stop-controlled or signal-controlled. If a horizontal curve on the ramp is relatively close to the intersection, a design speed for the curve should be selected which is appropriate for expected operations at the curve. For such a curve, the radius will determine whether the open-road or turning-roadway condition applies. For $R \geq 100$ m, use the open-road condition. For $R < 100$ m, the open-road condition is desirable, but the turning-roadway condition is acceptable.

48-5.03(07) Ramp Terminus (Merge Control)

An interchange ramp may terminate with a merge into the intersecting road. The horizontal alignment at the ramp merge (or junction) will be based on the open-road condition. The profile of a ramp terminus should be designed with a platform on the ramp side of the approach nose or merging end. The platform should be at least 60 m in length. It should have a profile that does not greatly differ from that of the adjacent traffic lane.

48-5.04 Vertical Alignment

48-5.04(01) Grade

The maximum grade for vertical alignment on a ramp cannot be as definitively expressed as that for the highway mainline. The value of the limiting grade is 3% to 5%, but, for a given ramp, the selected grade is dependent upon the following.

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

2. The steepest grade should be designed for the longitudinal center portion of the ramp. A freeway-ramp junction or landing area at an at-grade intersection should be as flat as practical.

3. A short upgrade of as much as 5% does not interfere with truck or bus operations. Consequently, for new construction it is desirable to limit the maximum grade to 5%.

4. A downgrade on a ramp should be in accordance with the same guidelines as those for an upgrade. It may, however, safely exceed the value 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.

Design Speed, km/h	30-40	40-50	60	70-80
Desirable Maximum Grade, %	6 to 8	5 to 7	4 to 6	3 to 5

48-5.04(02) Vertical Curve

A vertical curve on a ramp should be designed the same as that on the mainline. At a minimum, it should be designed to be in accordance with 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. 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 Forty-four for details on the design of a vertical curve.

<u>48-5.05 Roadside Safety</u>

The criteria described in Chapter Forty-nine (e.g., clear zone, barrier warrants) will apply to the roadside-safety design of an interchange ramp. A median barrier is required between adjacent on- and off-ramps (e.g., between the outside directional ramp and inside loop ramp of a cloverleaf interchange). This situation occurs at a full or partial-cloverleaf interchange.

48-6.0 OTHER INTERCHANGE-DESIGN CONSIDERATIONS

<u>48-6.01 General Considerations</u>

1. <u>Design Year</u>. The design year for the minor road intersecting the freeway should be the same as that used for the freeway. The termination of other roads and streets in the area may generate a significant increase of traffic on the crossing facility.

2. Overpass versus Underpass. The decision on whether the freeway should overpass or underpass the crossroad is dictated by topography. If the topography does not favor one or the other, the following 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. Considerations include the amount of fill, grading, structure span lengths, angle of skew, grades, sight distance, geometrics, constructability, traffic control, or costs.

   b. One benefit of the crossroad overpassing the freeway is that this may improve the ramp grades. As a driver exits the freeway, he or she will tend to slow down while using an exit ramp or speed up while using an entrance ramp.

   c. The alternative which provides the highest design level for the major road should be selected. The crossroad has a lower design speed; therefore, the minor road typically can be designed with steeper grades, lesser width, reduced vertical clearance requirements, etc.

   d. If a crossing or structure is planned for a future date, the mainline should underpass such future crossing. An overpass is easier to install and will be less disruptive to the major road once it is constructed in the future.

3. Underpass Width. The approach cross section, 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. A wider underpass also provides greater sight distance for an at-grade ramp terminal near the structure.

4. Grading. The designer should consider the grading around an interchange early in the design. A properly-graded interchange allows the overpass structure to naturally blend into the terrain. The slopes should not be too steep to support the bridge and roadway. It should be able to support plantings which prevent erosion and enhance the appearance of the area. Flatter slopes also allow for easier maintenance.

48-6.02 Freeway-Lane Drop

A freeway-lane drop, where the basic number of lanes is decreased, should occur on the freeway mainline away from other turbulence such as an interchange exit or entrance. However, it may be advantageous to drop a basic freeway lane at a 2-lane exit.

Figure 48-6A illustrates the design of a lane drop beyond an interchange. The following should also be considered.

1. Location. The lane drop should occur approximately 600 to 900 m beyond the previous interchange. Under restricted conditions, the signing distance shown in the MUTCD is acceptable. This distance allows for adequate signing and driver adjustments from the interchange, but yet is not so far downstream that a driver becomes accustomed to the number of lanes and is surprised by the lane drop. A lane should not be dropped on a horizontal curve or where other signing is required, such as for an upcoming exit.

   In an urban area, interchanges may be closely spaced for a considerable length of highway. 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 at 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 volume for a significant length of freeway.

2. Transition. The transition taper length should be 70:1. The minimum taper rate that can be used is 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 the applicable DSD value. If determining the availability of DSD, the desirable height of object will be 0.0 m (the roadway surface). It is acceptable to use 150 mm. This criterion would favor, for example, placing a freeway-lane drop within a sag vertical curve rather than just beyond a crest vertical curve.

4. Right-Side versus Left-Side Drop. A right-side freeway-lane drop is preferred. However, a left-side lane reduction may be more practical at a specific location (e.g., where it is planned to continue the left lane in the median in the future.

5. Shoulder. 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 3.0 m (or 3.6 m) to 1.2 m. The full 3.0-m left shoulder should be maintained for a distance of approximately 90 m beyond the merge point of the dropped lane. This provides an area to allow a driver, who may have missed the signing, an opportunity to safely merge with the through traffic. A full-depth shoulder pavement needs to be provided for 90 m beyond the merge point.

48-6.03 Collector-Distributor Road

An interchange that is designed with a single exit is superior to that designed 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.

A collector-distributor (C-D) road uses the single-exit approach to improve the interchange operational characteristics. A C-D road will improve the interchange as follows:

1. remove weaving maneuvers from the mainline and transfer them to the slower speed C-D road;

2. provide a high-speed single exit or entrance 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.

A C-D road is most often warranted if traffic volume is so high that the interchange without the C-D road cannot operate at an acceptable level of service, especially in a weaving area. It is particularly advantageous at a full-cloverleaf interchange where the weaving between the ramp and mainline traffic can be very difficult. Figure 48-2E illustrates a schematic of a C-D within a full-cloverleaf interchange.

A C-D road may be one or two lanes, depending upon the traffic volume 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 ranges from 70 to 80 km/h. However, it should be within 20 km/h of the mainline design speed. The separation between the C-D road and mainline should be as wide as practical but not less than that required to provide the applicable shoulder widths and a longitudinal barrier between the two (e.g., 6.0 to 7.8 m).

48-6.04 Frontage Road

The designer must consider the impact of a frontage road, where present, on interchange design. At an urban interchange, it may be impractical to separate the intersection of the ramp and frontage road with the crossing road. 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 a restricted urban area. The critical design element is the distance A between the ramp-frontage road merge and the crossroad. 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 provides 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 speed, and intersection queue distance. Therefore, a detailed analysis will be necessary to firmly establish the needed distance to properly accommodate vehicular operation. Additional information can be found in Transportation Research Record 682, *Distance Requirements for Frontage-Road Ramps to Cross Streets: Urban Freeway Design*.
Distance B shown in Figure 48-6B should be determined based on the number of frontage road lanes and the intersection design. This distance is determined as the weave distance from the intersection to the ramp entrance. For capacity analysis of the weave section, see the *Highway Capacity Manual*. This distance may be 0.0 m.

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

1. Slip Ramp. A slip ramp may be used to connect the freeway with a one-way frontage road before or after the intersection with the crossroad. A newly-constructed slip ramp to a 2-way frontage road is unacceptable because it may induce wrong-way entry onto the freeway and may cause an accident at the intersection of the ramp and frontage road.

2. Separate Intersections. Separate ramp-crossroad and frontage road-crossroad intersections may be accomplished by curving the frontage road away from the ramp and intersecting the frontage road with the crossroad outside the ramp limits of full access control. Figure 48-6D 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 design criteria for a frontage road (e.g., functional classification, outer separation).

48-6.05 Ramp-Crossroad Intersection

At a service interchange, the ramp will end with an at-grade intersection at the crossroad. The intersection should be treated as described in Chapter Forty-six. 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. Considerations to be made in the design of a ramp-crossroad intersection are as follows.

1. Capacity. In an urban area where traffic volume is high, inadequate capacity of the ramp-crossroad intersection can adversely affect the operation of the ramp-freeway junction. The safety and operation of the mainline itself may be impaired by a backup onto the freeway. Therefore, sufficient capacity and storage space should be provided for an at-grade

intersection or a merge with the crossroad. This may require adding additional lanes at the intersection or on the ramp proper, or it may 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 intersection sight distance. The criteria also apply to a ramp-crossroad intersection. The location of the bridge pier, abutment, sidewalk, bridge rail, roadside barrier, etc., should be considered. These may present major sight-distance obstacles. The bridge and the required intersection sight distance may result in the need to relocate the ramp-crossroad intersection.

3. Wrong-Way Movement. A wrong-way movement may originate at the ramp-crossroad intersection. The intersection must be properly signed and designed to minimize the potential for a wrong-way movement (e.g., channelization).

4. Turn Lane. An additional turn lane may be required at the end of ramp. Section 46-4.0 provides information on the design of a turn lane at an at-grade intersection.

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. Upon leaving a ramp, a driver 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 an exiting driver to leave the through lanes without undue hindrance to through traffic.

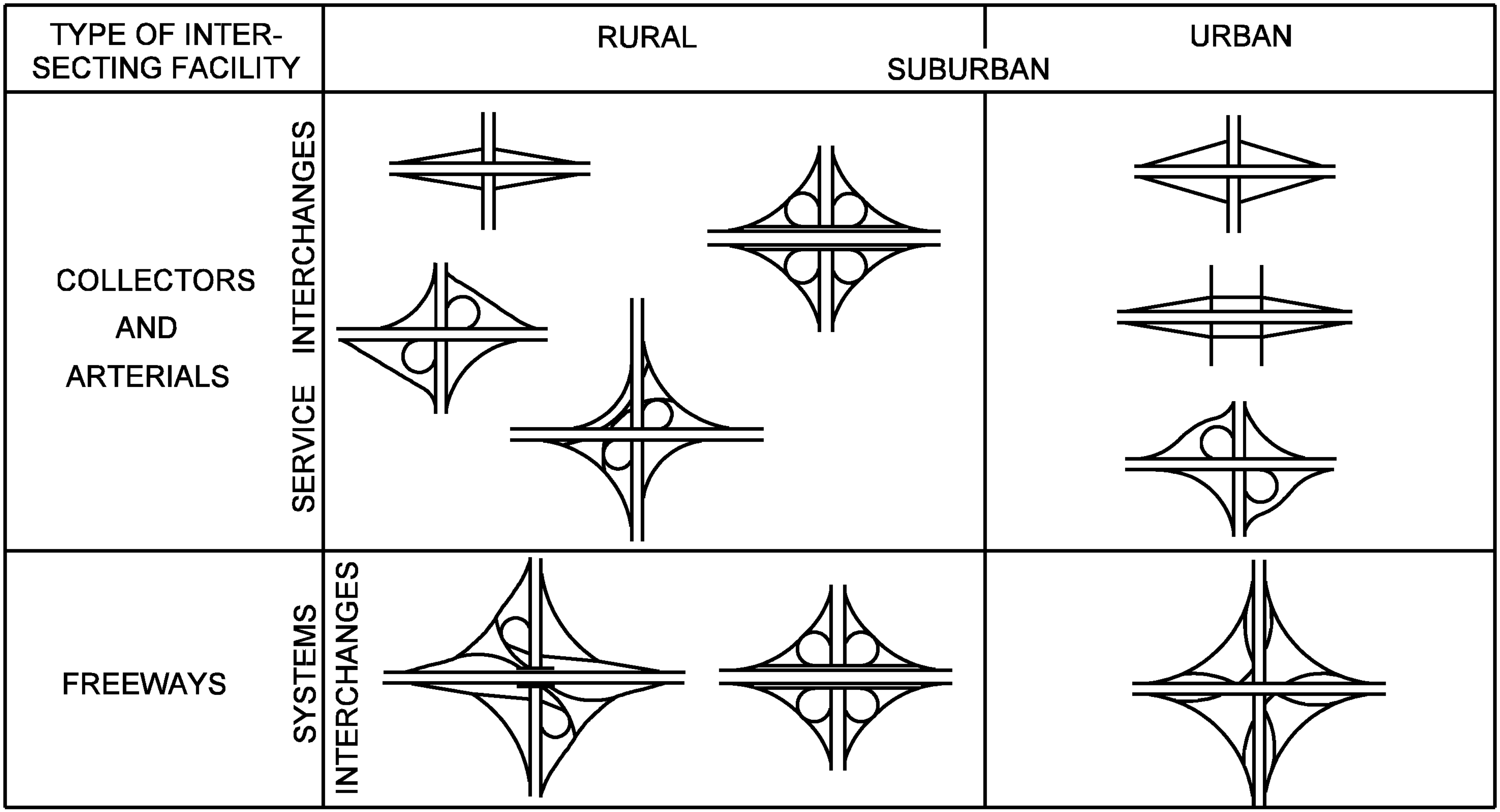
48-6.06 Access Control

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

Figures 48-6C, 48-6D, and 48-6E illustrate the access control for typical interchange designs. These figures provide the location of the full-access-control lines along a ramp, at a ramp-crossroad intersection, across from a ramp terminal, and along a frontage road.

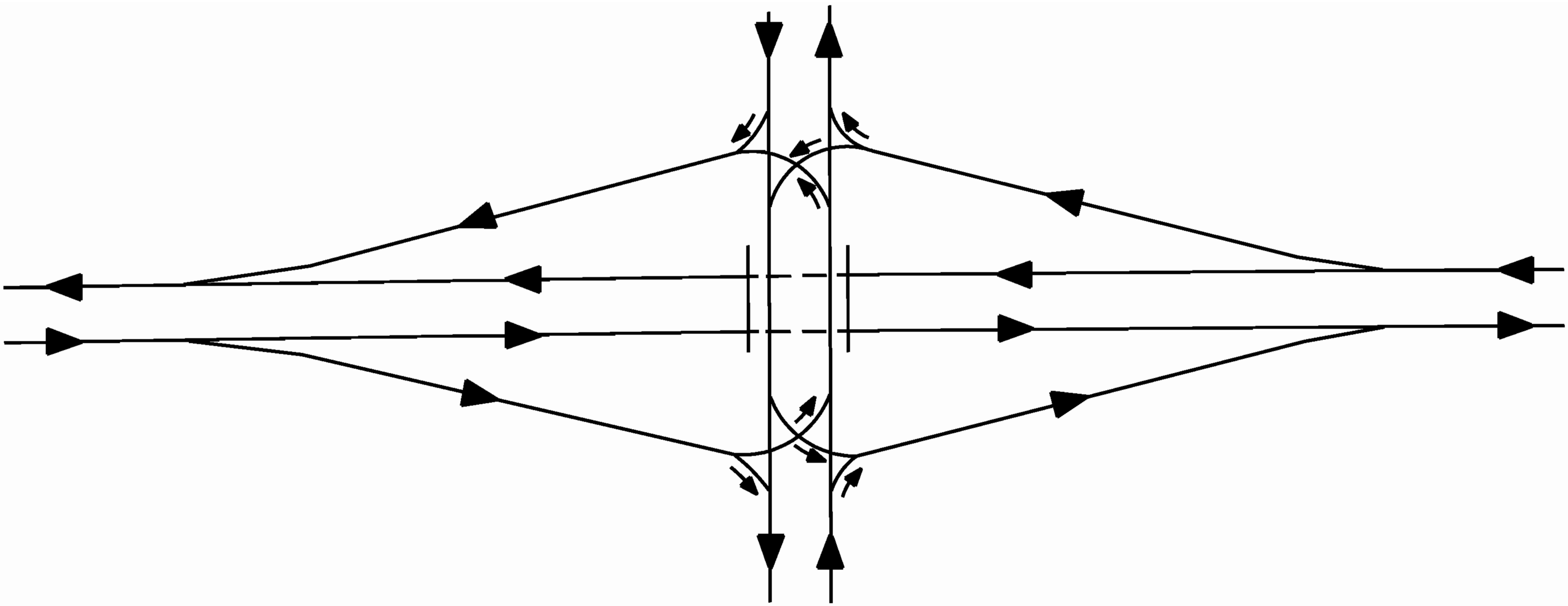
As indicated in the figures, the full-access-control lines extend a distance along the crossroad beyond the ramp or frontage road taper extremity on both sides of the road. The 30 m to 60 m in an urban area, or the 90 m to 150 m in a rural area should satisfy congestion concerns. However, in an area where the potential for development exists that would create traffic problems, it may be appropriate to consider a longer length of access control. An area may have changed over the years from rural to urban. As indicated, the Department has adopted different criteria for access control at an urban or a rural interchange. However, a change in area character alone is not a sufficient justification to alter the location of the full-access-control line where an existing interchange will be rehabilitated or if INDOT receives requests for additional access points from outside interests.

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



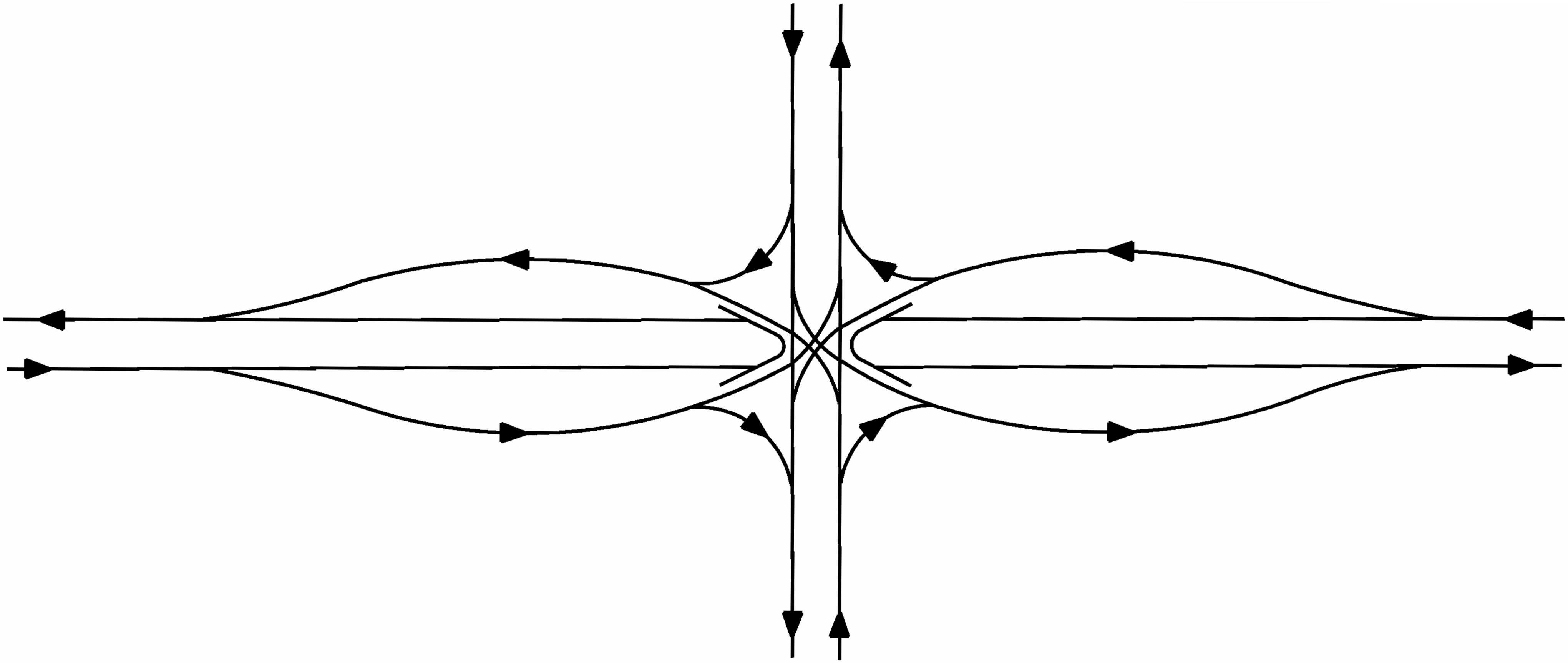
FREEWAY INTERCHANGES
(Based on Functional Classification of Intersecting Facility)

Figure 48-2A



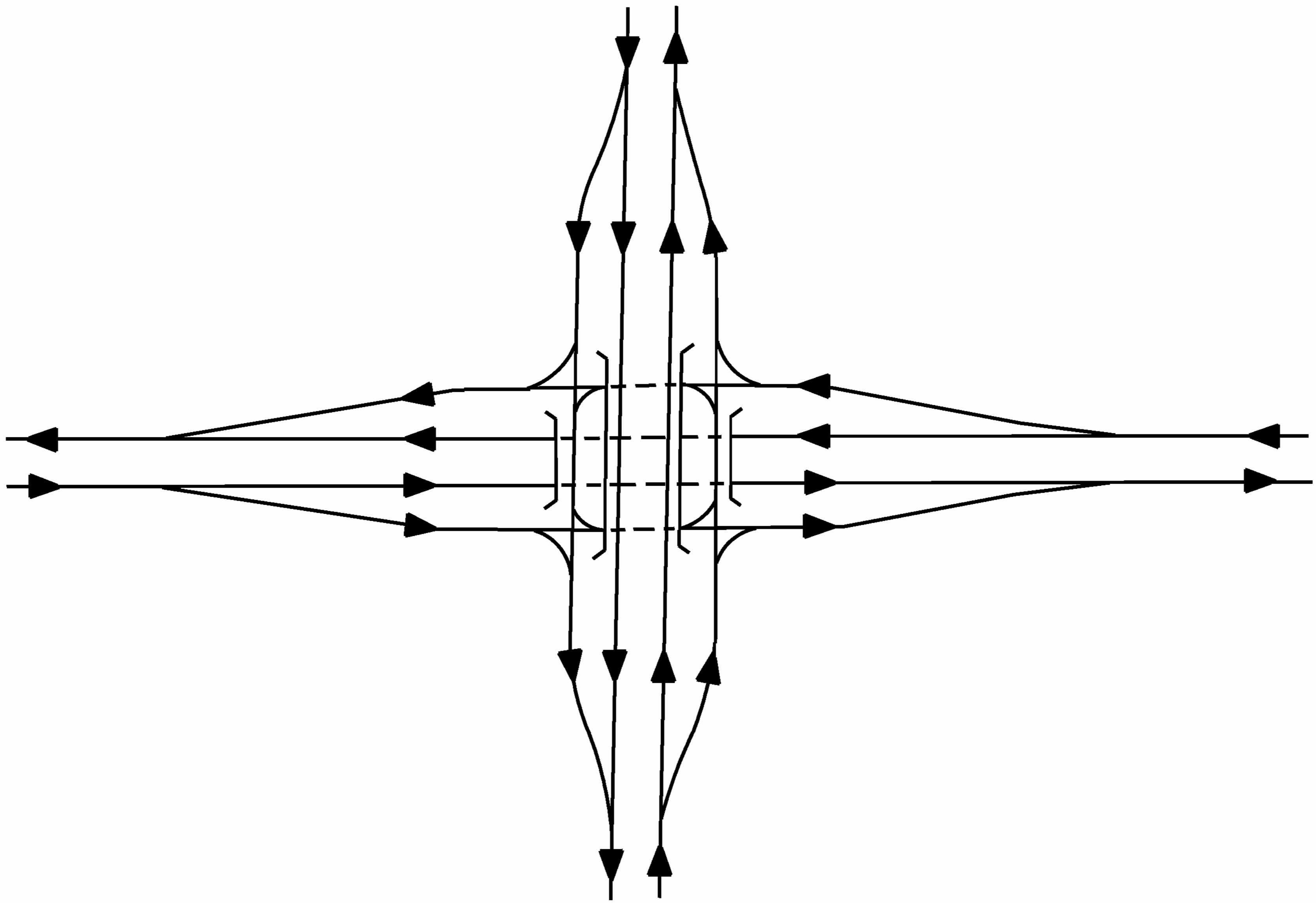
DIAMOND INTERCHANGE

Figure 48-2B



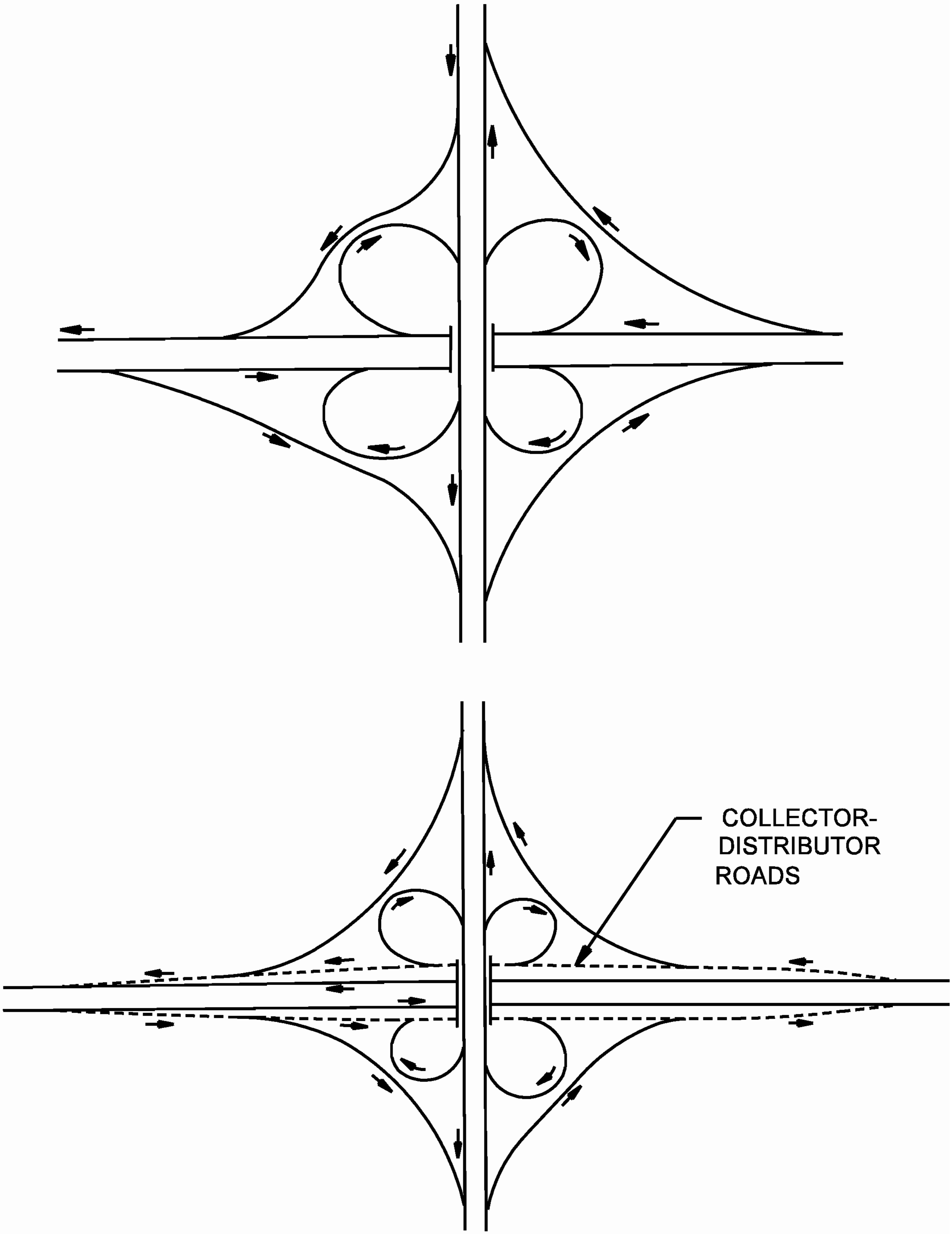
SINGLE POINT URBAN INTERCHANGE

Figure 48-2C



THREE-LEVEL DIAMOND INTERCHANGE

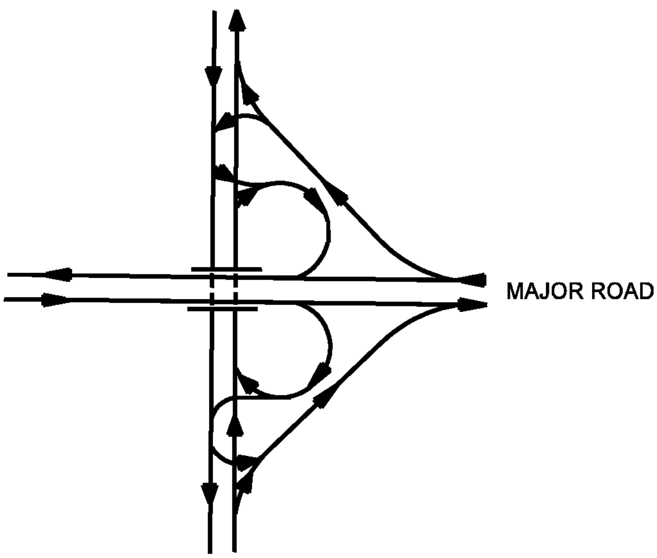
Figure 48-2D



FULL CLOVERLEAFS

Figure 48-2E

-A-

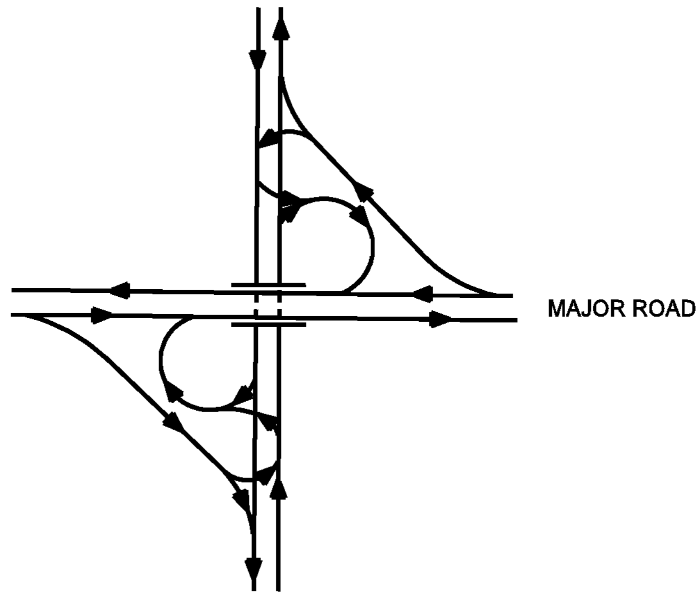


ON BOTH SIDES OF MAJOR ROAD

LEFT TURNS: NONE ON MAJOR ROAD
FOUR ON MINOR ROAD

TWO QUADRANTS ADJACENT

-B-

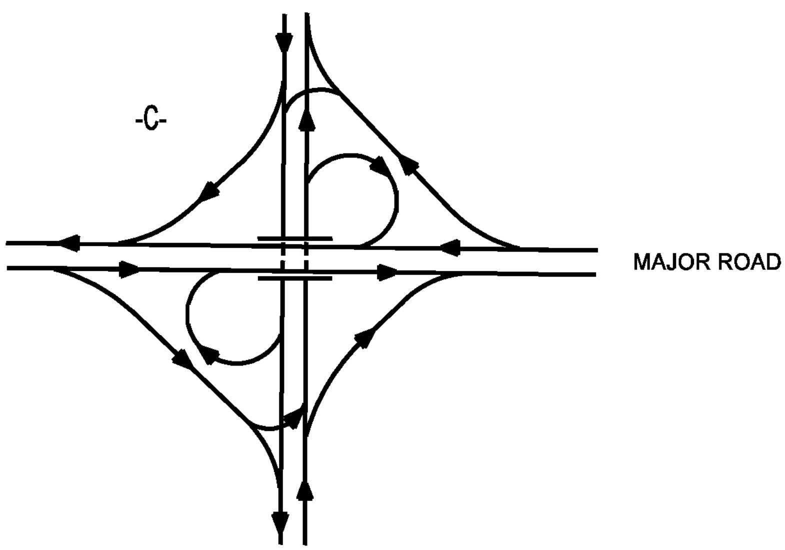


MAJOR ROAD EXITS ON NEAR SIDE

LEFT TURNS: NONE ON MAJOR ROAD
FOUR ON MINOR ROAD

TWO QUADRANTS
DIAGONALLY OPPOSITE

-C-



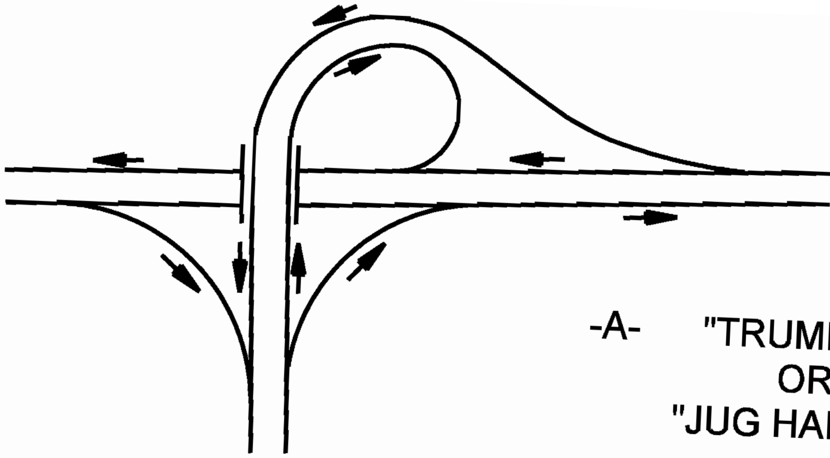
MAJOR ROAD EXITS ON NEAR SIDE

LEFT TURNS: NONE ON MAJOR ROAD
TWO ON MINOR ROAD

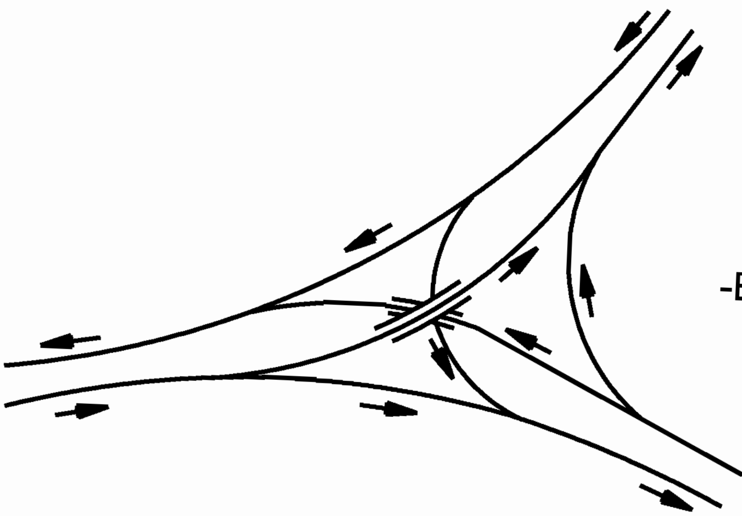
FOUR QUADRANTS

PARTIAL CLOVERLEAF ARRANGEMENTS

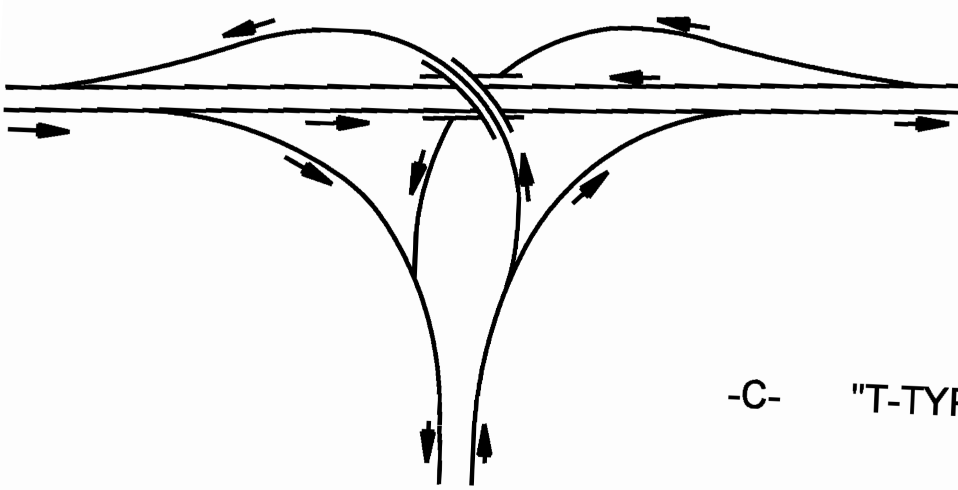
Figure 48-2F



-A- "TRUMPET" OR "JUG HANDLE"



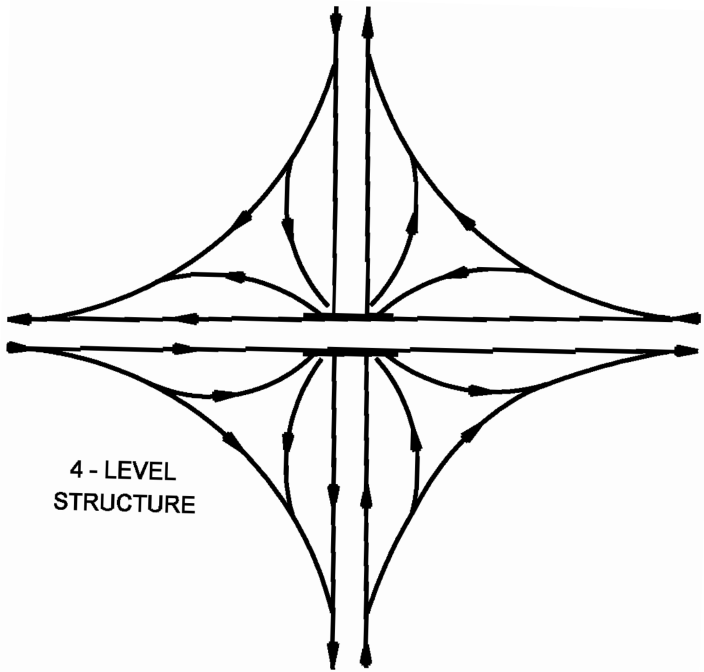
-B- "Y-TYPE"



-C- "T-TYPE"

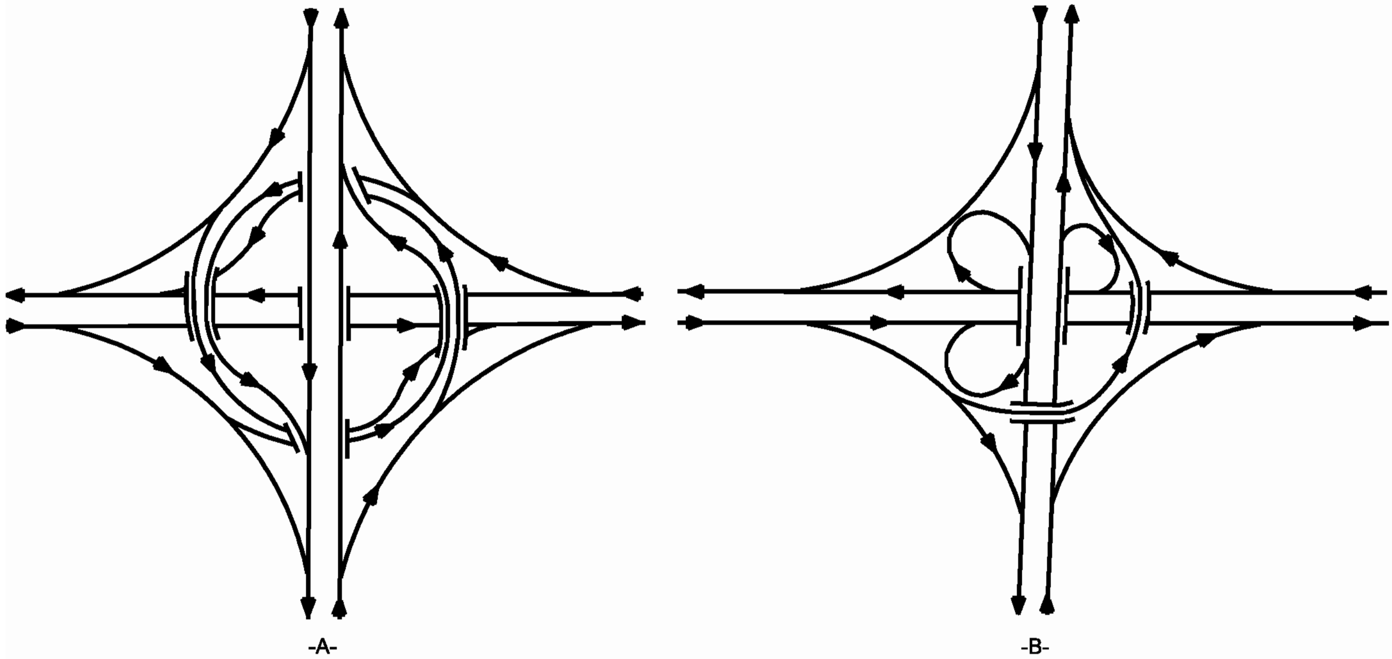
THREE-LEG INTERCHANGE

Figure 48-2G



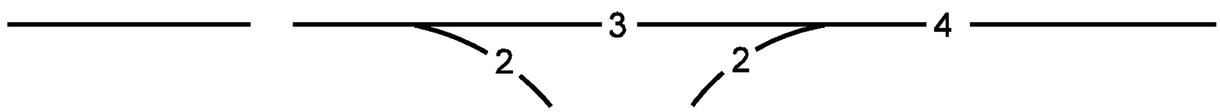
FULLY DIRECTIONAL INTERCHANGE

Figure 28-2H



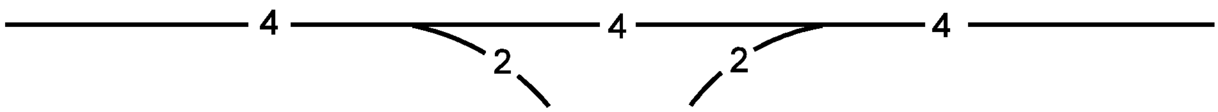
SEMI-DIRECTIONAL INTERCHANGES

Figure 28-2I



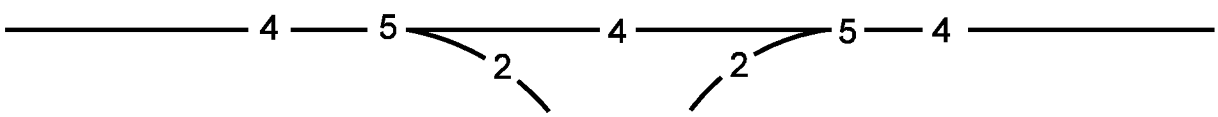
LANE BALANCE BUT NO COMPLIANCE WITH BASIC NUMBER OF LANES

-A-



NO LANE BALANCE BUT COMPLIANCE WITH BASIC NUMBER OF LANES

-B-

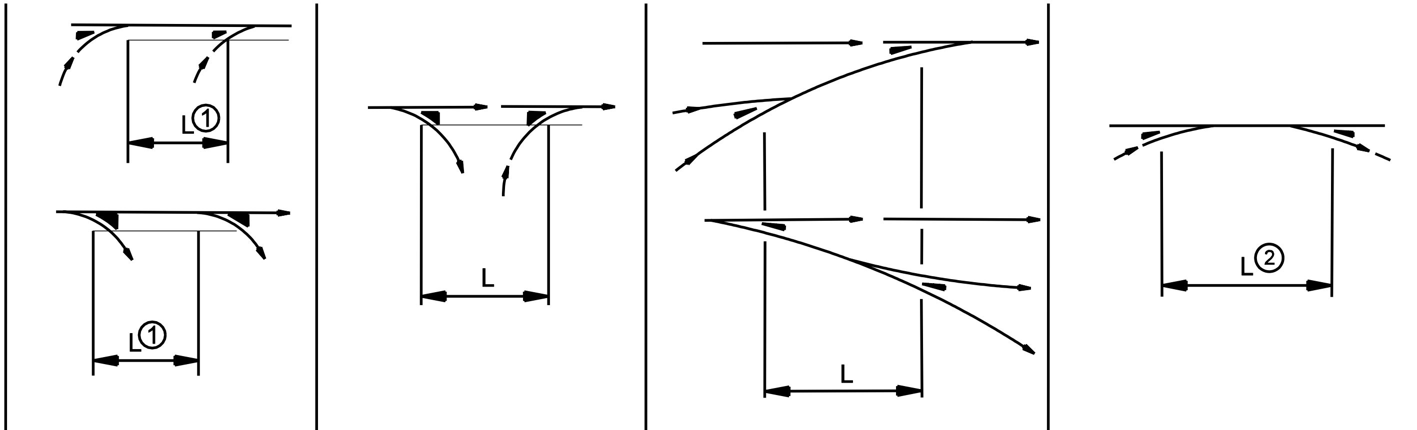


COMPLIANCE WITH BOTH LANE BALANCE AND BASIC NUMBER OF LANES

-C-

COORDINATION OF LANE BALANCE AND BASIC NUMBER OF LANES

Figure 48-3A



FULL FREEWAY	CDR OR FDR	FULL FREEWAY	CDR OR FDR	SYSTEM INTER-CHANGE	SERVICE INTER-CHANGE	SYSTEM TO SERVICE INTERCHANGE		SERVICE TO SERVICE INTERCHANGE	
EN-EN	EX-EX					FULL FWY.	CDR OR FDR	FULL FWY.	CDR OR FDR
MINIMUM LENGTHS (L) MEASURED BETWEEN SUCCESSIVE RAMP TERMINALS (m)									
300	240	150	120	240	180	600	480	480	300

NOTE: FDR - FREEWAY DISTRIBUTOR ROAD
CDR - COLLECTOR-DISTRIBUTOR ROAD

EN - ENTRANCE
EX - EXIT

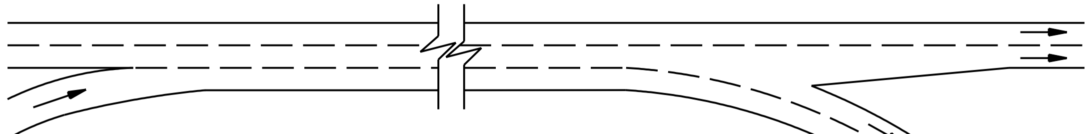
① DISTANCE IS MEASURED FROM THE END OF TAPER TO THE GORE AND DESIRABLY SHOULD BE 200 m OR MORE.

② FOR CLOVERLEAF LOOP RAMPS THIS DISTANCE SHOULD DESIRABLY BE 300 m.

The recommendations are based on operational experience and need for flexibility and adequate signing. They should be checked in accordance with the procedure outlined in the Highway Capacity Manual and Chapter Forty-one. The larger of the values is suggested for use. Also, a procedure for measuring the length of he weaving section is given in Chapter 4 of the Highway Capacity Manual.

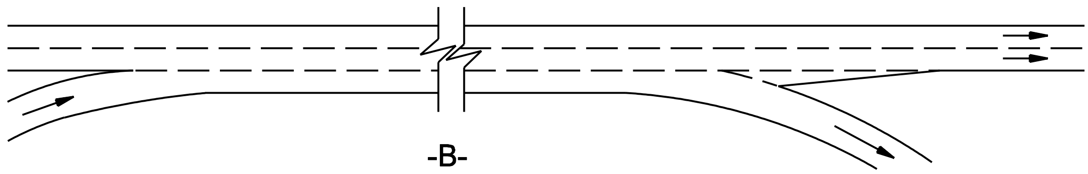
RECOMMENDED MINIMUM RAMP TERMINAL SPACING

Figure 48-3B



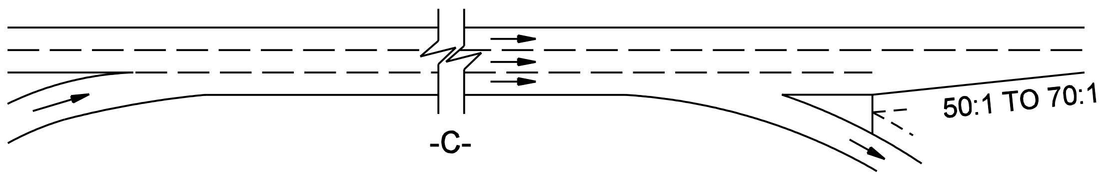
-A-

AUXILIARY LANE DROPPED ON EXIT RAMP



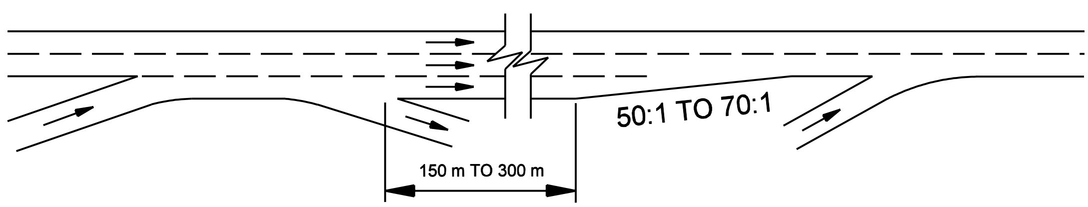
-B-

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



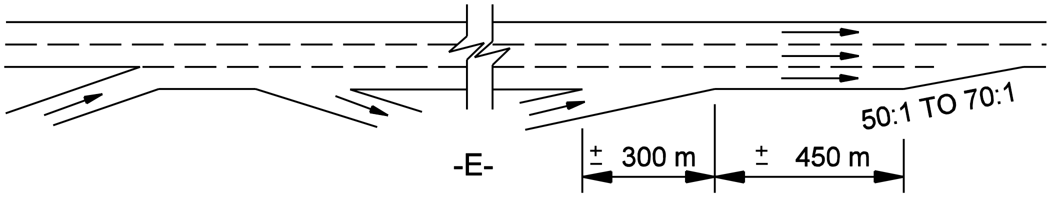
-C-

AUXILIARY LANE DROPPED AT PHYSICAL NOSE



-D-

AUXILIARY LANE DROPPED WITHIN AN INTERCHANGE

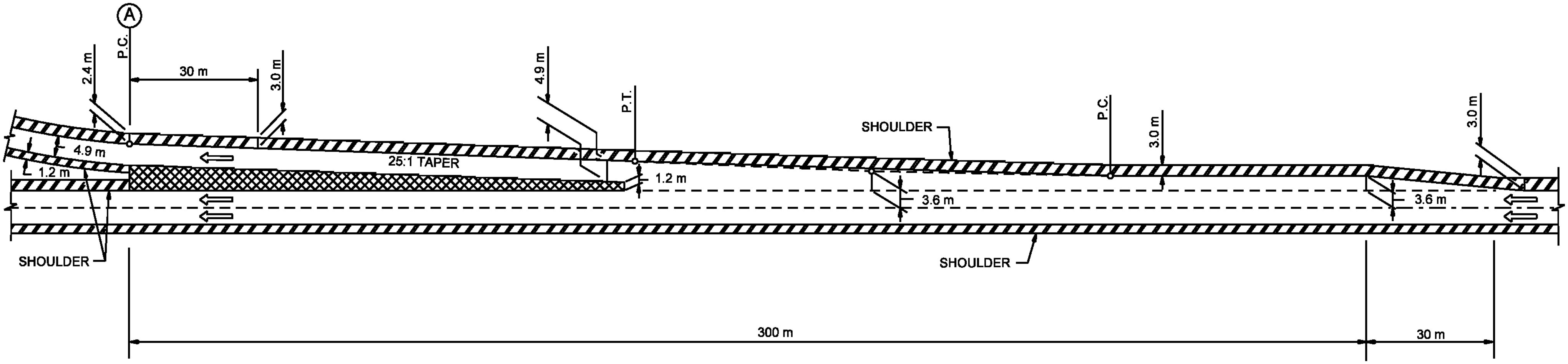


-E-

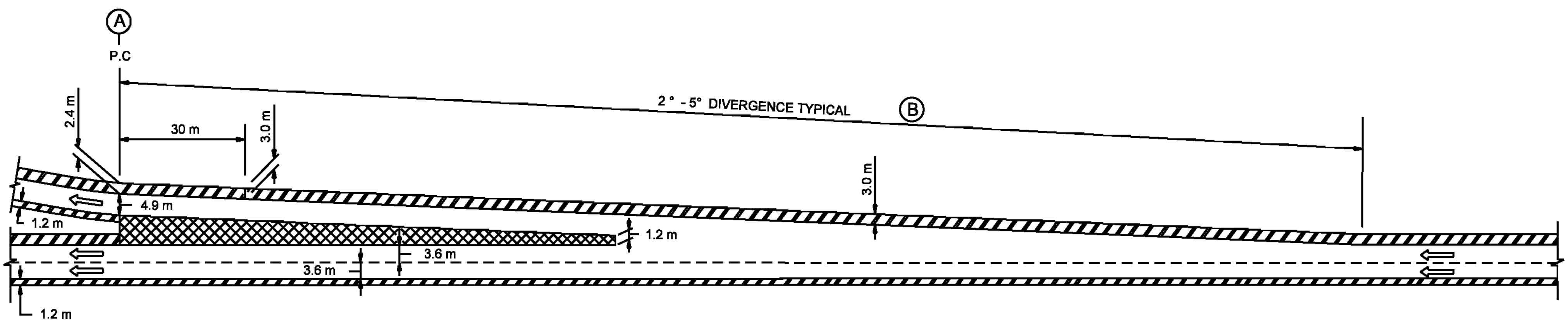
AUXILIARY LANE DROPPED BEYOND AN INTERCHANGE

ALTERNATE METHODS OF DROPPING AUXILIARY LANES

Figure 48-3D



PARALLEL DESIGN (PREFERRED)



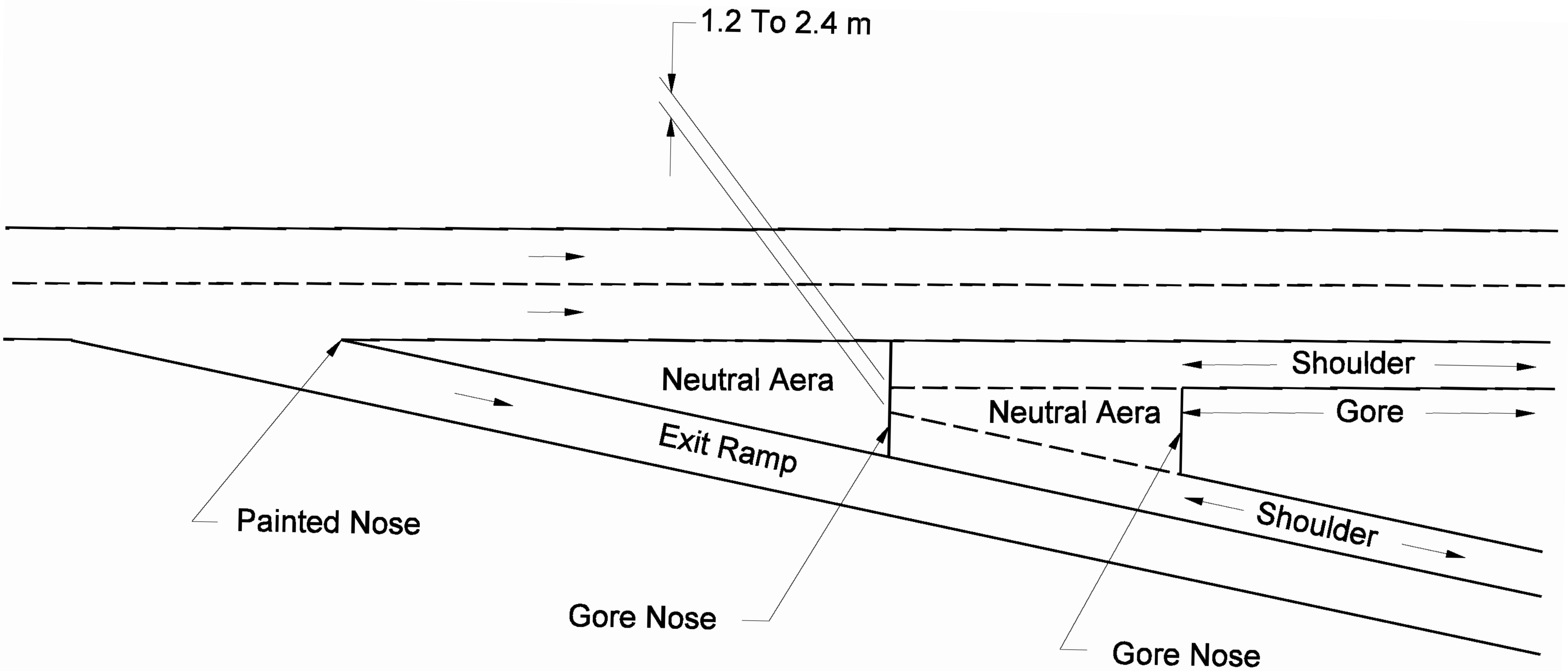
TAPERED DESIGN

A *Point controlling safe speed at ramp.*

B *See AASHTO A Policy on Geometric Design of Highways and Streets for applicable deceleration distances.*

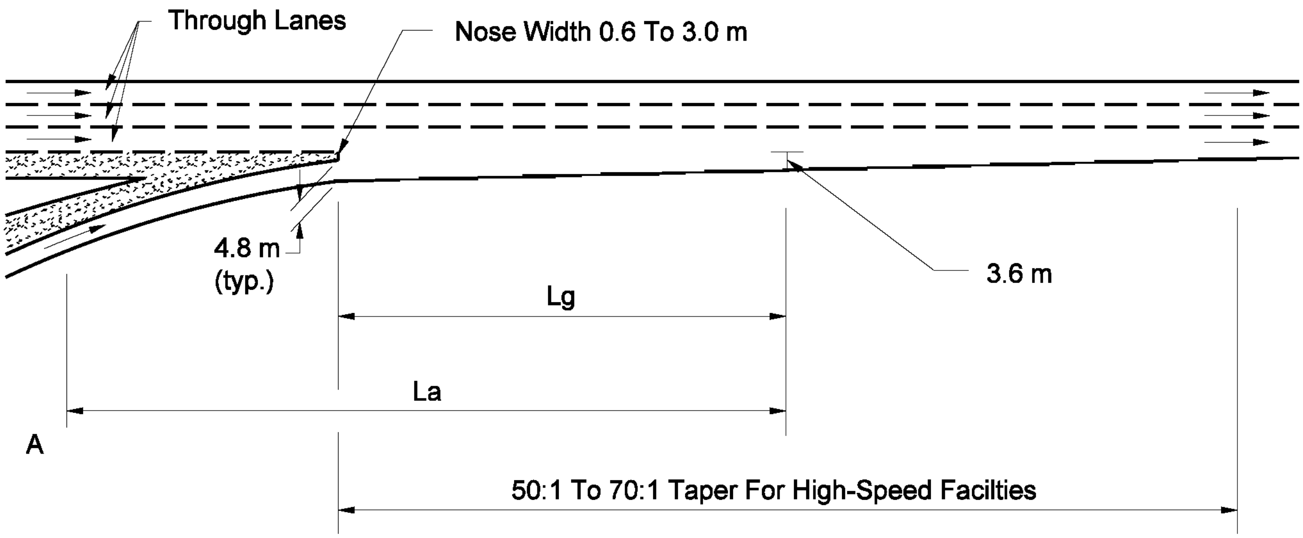
TYPICAL EXIT RAMP TYPES
(Single Lane)

Figure 48-4A

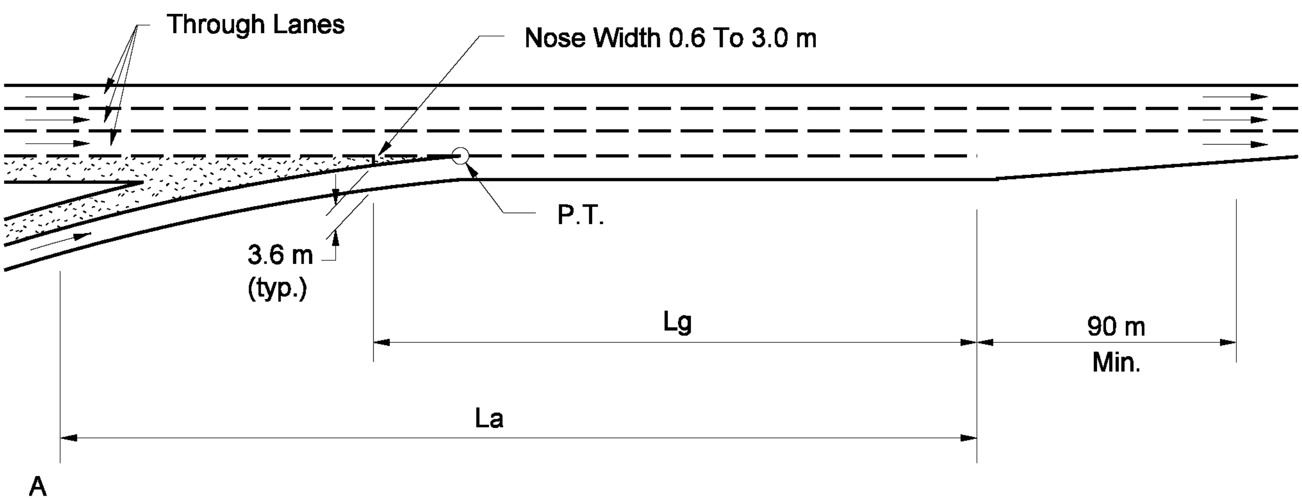


TYPICAL GORE AREA CHARACTERISTICS

Figure 48-4B



(A) Tapered Design



(B) Parallel Design

Notes:

1. La is the required acceleration length as shown in Figure 48-4D or as adjusted by Figure 48-4E.

2. Point A controls ramp speed. La should not start back on the curvature of the ramp unless the radius is 300 m or greater.

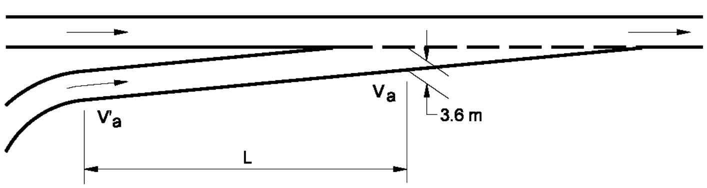
3. Lg is required gap acceptance length. Lg should be a minimum of 90 to 150 m, depending on the nose width.

4. The value of La or Lg, whichever produces the greater distance downstream from where the nose width equals 0.6 m, is suggested for use in the design of the ramp entrance.

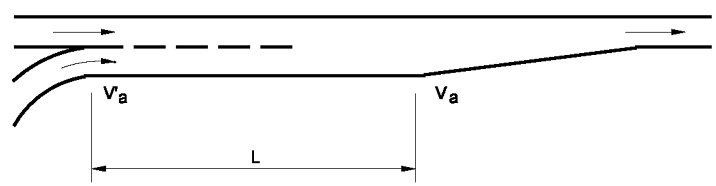
TYPICAL SINGLE-LANE ENTRANCE RAMPS

Figure 48-4C

Highway Design Speed (km/h)	Speed Reached, V a (km/h)	Acceleration Length, L (m) for Entrance Curve Design Speed (km/h)							
		Stop	20	30	40	50	60	70	80
		and Initial Speed, V'a (km/h)							
		0	20	28	35	42	51	63	70
50	31	60	50	--	--	--	--	--	--
60	39	95	80	65	--	--	--	--	--
70	47	150	130	110	90	65	--	--	--
80	54	200	180	165	145	115	65	--	--
90	61	260	245	225	205	175	125	35	--
100	69	345	325	305	285	255	205	110	40
110	75	430	410	390	370	340	290	200	125



TAPER TYPE



PARALLEL TYPE

Note: Uniform 50:1 to 70:1 tapers are recommended where lengths of acceleration lane exceed 400 m.

Minimum Acceleration Lengths for Entrance Terminals
With Flat Grades of 2 Percent or Less

Figure 48-4D

Highway Design Speed (km/h)	Ratio of Length on Grade to Length on Level for Ramp Curve Design Speed (mph)					
	40	50	60	70	80	All Speeds
	3% ≤ Upgrade < 4%					3% ≤ Downgrade < 4%
60	1.30	1.40	1.40	--	--	0.70
70	1.30	1.40	1.40	1.50	--	0.65
80	1.40	1.50	1.50	1.50	1.60	0.650
90	1.40	1.50	1.50	1.50	1.60	0.60
100	1.50	1.60	1.70	1.70	1.80	0.600
110	1.50	1.60	1.70	1.70	1.80	0.600
	4% ≤ Upgrade ≤ 6%					4% ≤ Downgrade ≤ 6%
60	1.50	1.50	--	--	--	0.60
70	1.50	1.60	1.70	--	--	0.60
80	1.50	1.70	1.90	1.80	--	0.55
90	1.60	1.80	2.00	2.10	2.20	0.55
100	1.70	1.90	2.20	2.40	2.75	0.50
110	2.00	2.20	2.60	2.80	3.00	0.50

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 - 110 km/h
Entrance Ramp Curve Design Speed - 70 km/h
Average Grade - 4.5% upgrade

Problem: Determine length of acceleration lane.

Solution: Figure 48-4D yields an acceleration length of 200 m on the level. According to the table shown above, this should be increased by the average of the increases (shown for 2.80).

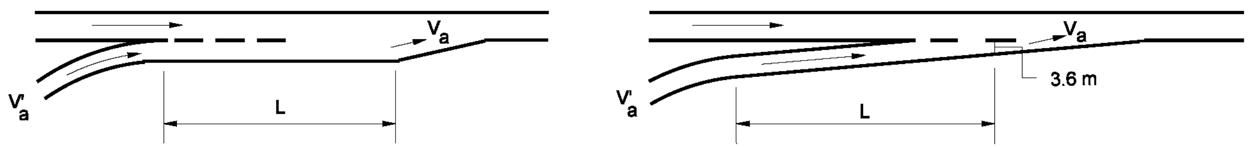
Therefore: L = (200 m)(2.80)
L = 560 m

An additional 515 m (560 m – 45 m) should be added to the ramp prior to the entrance taper. See Figure 48-4C, Typical Entrance Types.

GRADE ADJUSTMENT FOR ACCELERATION
(Passenger Car)

Figure 48-4E

Highway Design Speed (km/h)	Speed Reached, V'a (km/h)	L = Acceleration Length (m)						
		or Design Speed of First Governing Geometric Control (km/h)						
		Stop	30	40	50	60	70	80
		For Average Running Speed V'a (km/h)						
		0	40	47	30	55	63	70
50	31	50	--	--	--	--	--	--
60	39	75	35	--	--	--	--	--
70	47	150	120	80	--	--	--	--
80	54	265	235	200	130	--	--	--
90	61	480	455	425	365	175	--	--
100	69	730	705	675	615	425	260	--
110	75	1010	955	900	850	725	575	215



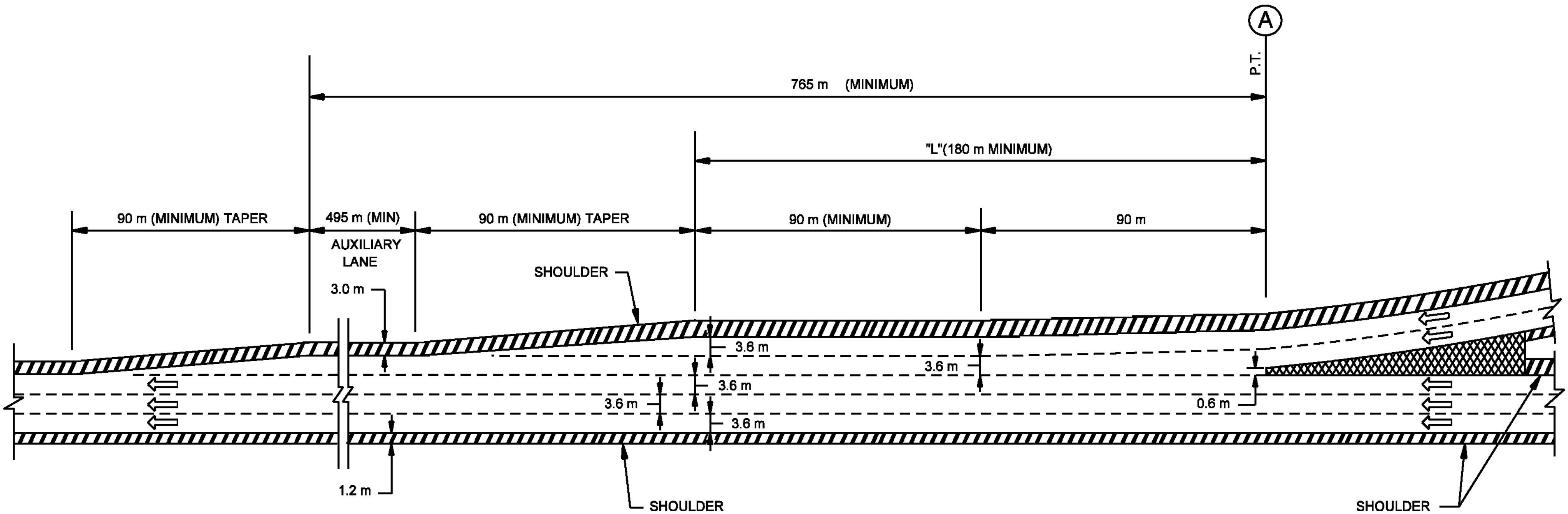
PARALLEL TYPE

TAPER TYPE

Note: The acceleration lengths are calculated from the distance needed for a 120 kg/kW truck to accelerate from the average running speed of the entrance curve to reach a speed (V'_a), which is 16 km/h below the average running speed on the mainline.

LENGTHS FOR ACCELERATION
(120 kg/kW Truck)

Figure 48-4F



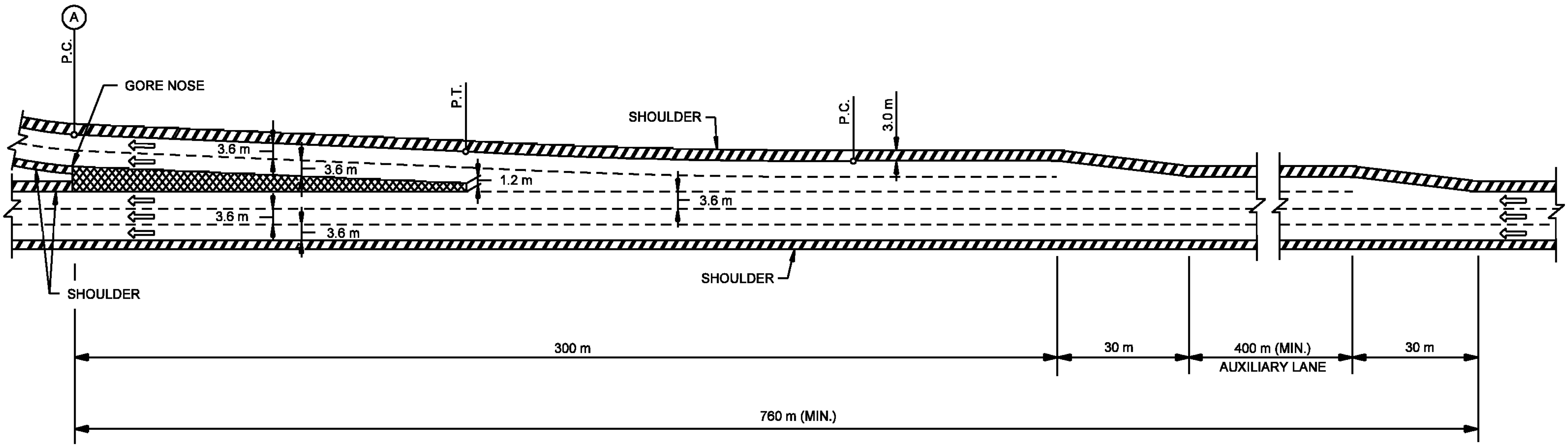
Notes:

1. *L is the required acceleration length as shown in Figures 48-4D and 48-4E.*
   L = 180 m minimum.

2. *A parallel-lane design should be used at multi-land entrances. The design details are similar to those for a single-land entrance. See Figure 48-4C and the INDOT Standard Drawings.*

MULTI-LANE ENTRANCE RAMP
(Service Interchanges)

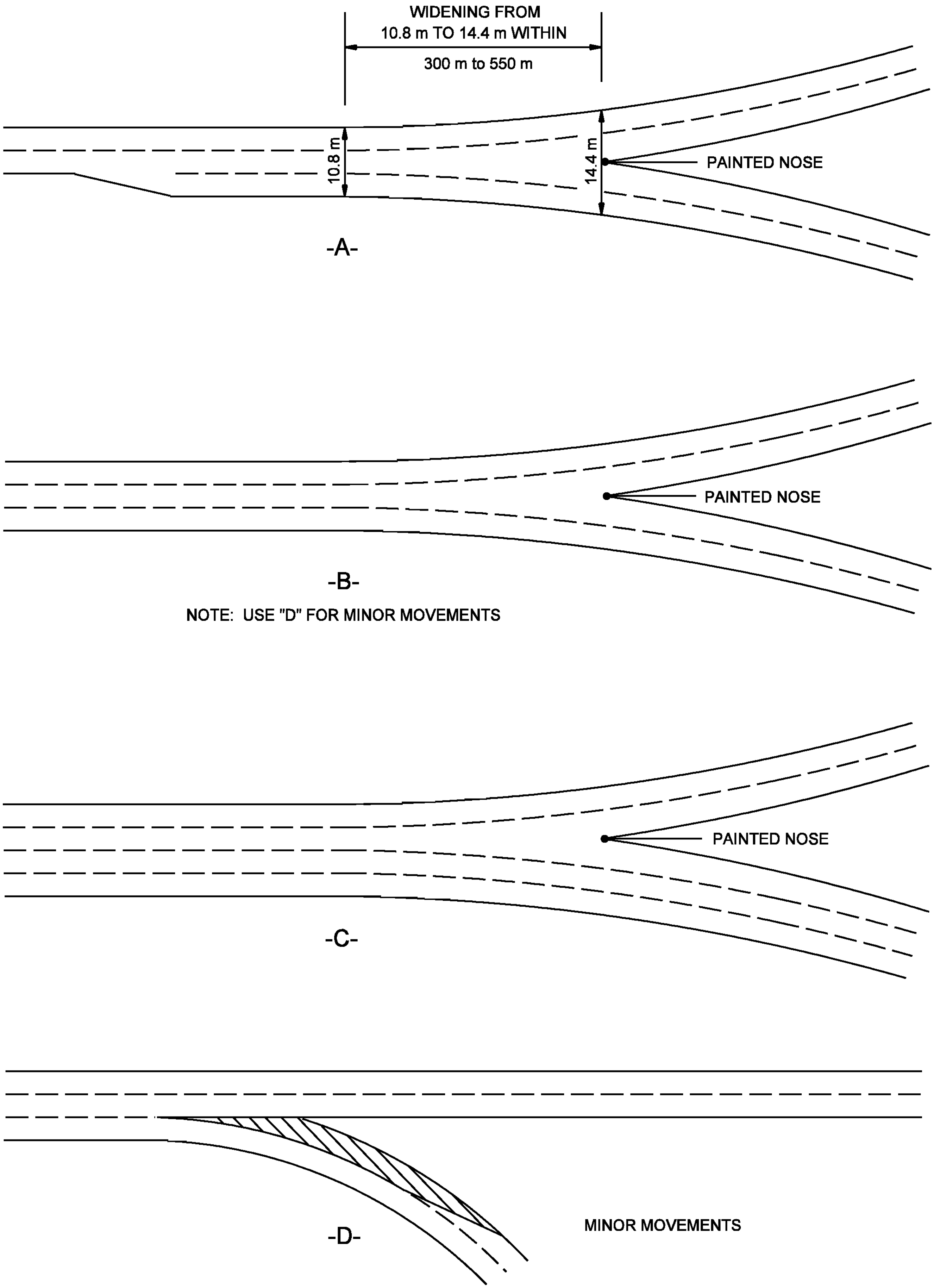
Figure 48-4G



Note: A parallel-lane design should be used at multi-lane exits. The desiign details are similar to those for a single-lane exit. See Figure 48-4A and the INDOT Standard Drawings.

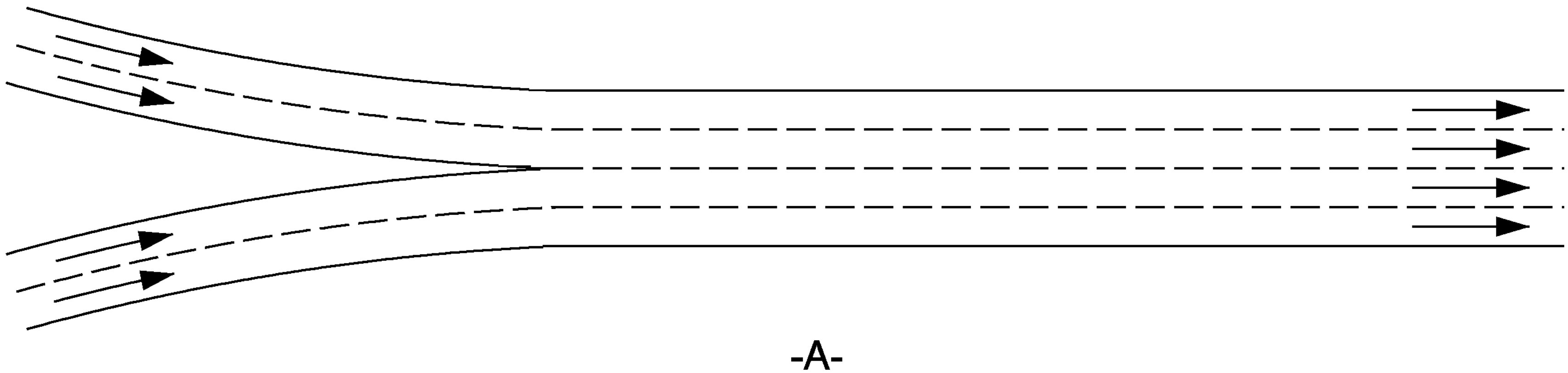
MULTI-LANE EXIT RAMP
(Service Interchanges)

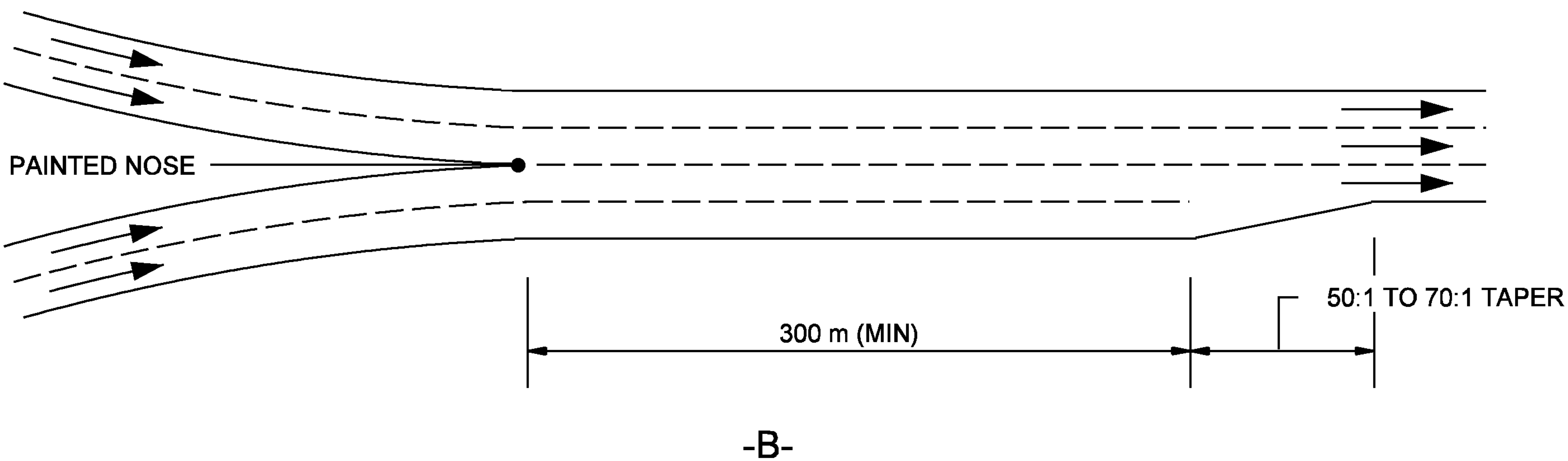
Figure 48-4H



MAJOR FORKS FOR SYSTEM INTERCHANGES
(Typical Schematics)

Figure 48-4I





BRANCH CONNECTIONS FOR SYSTEM INTERCHANGES
(Typical Schematics)

Figure 48-4J

Mainline Design Speed, km/h *	50	60	70	80	90	100	110	120
	Ramp Design Speed, km/h							
Upper Range, 85%	40	50	60	70	80	90	100	110
Middle Range, 70%	30	40	50	60	60	70	80	90
Lower Range, 50%	20	30	40	40	50	50	60	70
	Corresponding Minimum Radius, m							
All ranges	7	7	10	25	50	80	115	160

** Only a value for 80 km/h or higher may be applied to a freeway or expressway exit.*

RAMP DESIGN SPEED
AND CORRESPONDING MINIMUM RADIUS

Figure 48-5A

R *	V = 40			V = 50			V = 60			V = 70			V = 80			V = 90			V = 100		
	e	L		e	L		e	L		e	L		e	L		e	L		e	L	
		A	B		A	B		A	B		A	B		A	B		A	B		A	B
7000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
5000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
4000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
3000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	RC	15	20
2000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	RC	10	20	0.022	15	20	0.026	15	25
1500	NC	0	0	NC	0	0	NC	0	0	RC	10	20	0.024	15	20	0.028	20	30	0.034	20	35
1250	NC	0	0	NC	0	0	RC	10	15	0.023	15	20	0.028	15	25	0.033	20	30	0.040	25	40
1000	NC	0	0	RC	10	15	0.022	10	20	0.028	15	25	0.034	20	30	0.040	25	40	0.048	30	50
900	NC	0	0	RC	10	15	0.024	10	20	0.031	15	25	0.037	20	35	0.044	25	40	0.052	30	50
800	NC	0	0	RC	10	15	0.027	15	20	0.034	20	30	0.041	25	35	0.048	30	45	0.057	35	55
700	RC	10	15	0.022	10	15	0.030	15	25	0.038	20	30	0.045	25	40	0.053	30	50	0.063	40	65
600	RC	10	15	0.026	10	20	0.034	15	25	0.043	20	35	0.051	25	45	0.060	35	55	0.069	40	70
500	0.022	10	15	0.030	15	20	0.039	20	30	0.049	25	40	0.058	30	50	0.067	40	65	0.076	45	75
400	0.027	10	20	0.036	15	25	0.047	20	35	0.057	30	45	0.066	35	55	0.075	45	70	0.080	50	80
350	0.030	15	20	0.040	20	30	0.051	25	40	0.062	30	50	0.071	35	60	0.079	45	75	R_{min} = 395		
300	0.034	15	25	0.045	20	30	0.056	25	40	0.067	35	55	0.076	40	65	R_{min} = 305					
250	0.040	15	25	0.051	20	35	0.062	30	45	0.073	35	60	0.078	40	65						
225	0.043	20	30	0.054	25	35	0.066	30	50	0.076	35	60	R_{min} = 230								
200	0.046	20	30	0.058	25	40	0.070	30	50	0.079	40	60									
175	0.050	20	30	0.062	25	40	0.074	35	55	R_{min} = 175											
150	0.054	20	35	0.067	30	45	0.078	35	55												
125	0.059	25	40	0.072	30	50	0.080	35	60												
100	0.065	25	40	0.078	30	50	R_{min} = 125														
90	0.069	25	45	0.079	30	55															
80	0.072	30	45	R_{min} = 80																	
70	0.075	30	45																		
60	0.078	30	50																		
	R_{min} = 50																				

Key:

R = Radius of curve, m
V = Ramp design speed, km/h
e = Superelevation rate
L = Length of superelevation runoff from remove crown to full super., m
A = L for 1-way ramp rotated about the centerline of travelway, m
B = L for 2-way ramp rotated about the centerline of travelway, m
NC = Normal crown
RC = Remove (adverse) crown

* For a curve radius which is between the listed values, use a straight-line interpolation to determine the superelevation rate.

RATE OF SUPERELEVATION FOR INTERCHANGE RAMPS, e_{max} = 8%
Figure 48-5B

R *	V = 40			V = 50			V = 60			V = 70			V = 80			V = 90			V = 100		
	e	L		e	L		e	L		e	L		e	L		e	L		e	L	
		A	B		A	B		A	B		A	B		A	B		A	B		A	B
7000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
5000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
4000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
3000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	RC	15	20
2000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	RC	10	20	0.021	15	20	0.025	15	25
1500	NC	0	0	NC	0	0	NC	0	0	RC	10	20	0.022	15	20	0.027	15	25	0.031	20	30
1250	NC	0	0	NC	0	0	RC	10	15	0.022	10	20	0.026	15	25	0.031	20	30	0.036	25	35
1000	NC	0	0	RC	10	15	0.021	10	15	0.026	15	20	0.031	20	30	0.036	20	35	0.042	25	45
900	NC	0	0	RC	10	15	0.023	10	20	0.028	15	25	0.034	20	30	0.039	25	40	0.045	30	45
800	NC	0	0	RC	10	15	0.025	15	20	0.031	15	25	0.036	20	30	0.042	25	40	0.049	30	50
700	RC	10	15	0.021	10	15	0.028	15	20	0.034	20	30	0.040	20	35	0.046	25	45	0.052	30	50
600	RC	10	15	0.024	10	20	0.031	15	25	0.038	20	30	0.043	25	40	0.050	30	45	0.056	35	55
500	0.021	10	15	0.028	15	20	0.035	15	25	0.042	20	35	0.048	25	40	0.054	30	50	0.059	35	60
400	0.025	10	15	0.033	15	25	0.040	20	30	0.047	25	40	0.053	30	45	0.059	35	55	R_{min} = 435		
350	0.028	10	20	0.035	15	25	0.043	20	30	0.050	25	40	0.056	30	50	0.060	35	55			
300	0.031	15	20	0.039	15	25	0.046	20	35	0.054	25	45	0.059	30	50	R_{min} = 335					
250	0.035	15	25	0.042	20	30	0.050	25	35	0.057	30	45	R_{min} = 250								
225	0.037	15	25	0.044	20	30	0.053	25	40	0.059	30	45									
200	0.039	15	25	0.047	20	30	0.055	25	40	0.060	30	50									
175	0.041	15	25	0.050	20	35	0.058	25	45	R_{min} = 195											
150	0.044	20	30	0.053	25	35	0.060	25	45												
125	0.047	20	30	0.056	25	40	R_{min} = 135														
100	0.052	20	35	0.060	25	40															
90	0.054	20	35	0.060	25	40															
80	0.056	20	35	R_{min} = 90																	
70	0.058	25	35																		
60	0.060	25	40																		
	R_{min} = 55																				

Key:

R = Radius of curve, m
V = Ramp design speed, km/h
e = Superelevation rate
L = Length of superelevation runoff from remove crown to full super., m
A = L for 1-way ramp rotated about the centerline of travelway, m
B = L for 2-way ramp rotated about the centerline of travelway, m
NC = Normal crown
RC = Remove (adverse) crown

* For a curve radius which is between the listed values, use a straight-line interpolation to determine the superelevation rate.

RATE OF SUPERELEVATION FOR INTERCHANGE RAMPS, e_{max} = 6%
Figure 48-5C

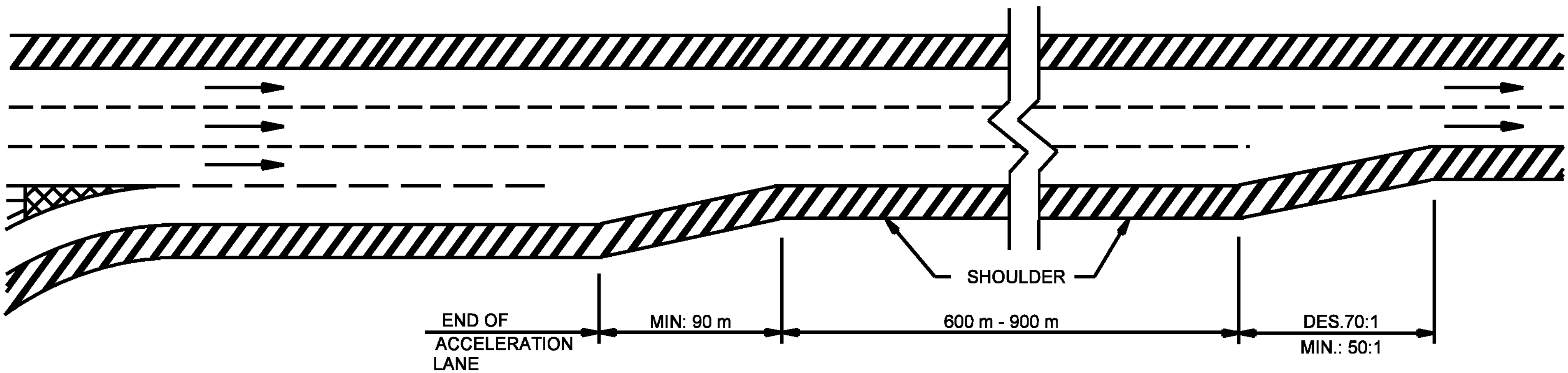
R *	V = 40			V = 50			V = 60			V = 70			V = 80			V = 90			V = 100		
	e	L		e	L		e	L		e	L		e	L		e	L		e	L	
		A	B		A	B		A	B		A	B		A	B		A	B		A	B
7000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
5000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
4000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
3000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	RC	15	20
2000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	RC	10	20	RC	15	20	0.022	15	25
1500	NC	0	0	NC	0	0	NC	0	0	RC	10	20	RC	10	20	0.023	15	25	0.026	15	25
1250	NC	0	0	NC	0	0	RC	10	15	RC	10	20	0.022	15	20	0.026	15	25	0.029	20	30
1000	NC	0	0	NC	0	0	RC	10	15	0.022	10	20	0.025	15	20	0.028	20	30	0.032	20	35
900	NC	0	0	RC	10	15	RC	10	15	0.024	15	20	0.027	15	25	0.030	20	30	0.034	20	35
800	NC	0	0	RC	10	15	0.021	10	15	0.025	15	20	0.028	15	25	0.032	20	30	0.035	25	35
700	NC	0	0	RC	10	15	0.023	10	20	0.027	15	25	0.030	15	25	0.034	20	35	0.037	25	40
600	RC	10	15	0.021	10	15	0.025	15	20	0.029	15	25	0.032	20	30	0.036	20	35	0.039	25	40
500	RC	10	15	0.023	10	15	0.027	15	25	0.031	15	25	0.035	20	30	0.038	25	35	0.040	25	40
400	0.021	10	15	0.025	10	20	0.029	15	25	0.034	20	30	0.037	20	35	0.040	25	40	R_{min} = 490		
350	0.023	10	15	0.027	15	20	0.031	15	25	0.036	20	30	0.039	20	35	R_{min} =375					
300	0.024	10	15	0.028	15	20	0.033	15	25	0.038	20	30	0.040	20	35						
250	0.026	10	20	0.030	15	20	0.036	15	30	0.039	20	30	R_{min} = 280								
225	0.027	10	20	0.032	15	25	0.037	20	30	0.040	20	30									
200	0.028	10	20	0.033	15	25	0.038	20	30	R_{min} = 215											
175	0.029	15	20	0.035	15	25	0.039	20	30												
150	0.031	15	20	0.037	15	25	0.040	20													
125	0.033	15	20	0.039	15	25	R_{min} = 150														
100	0.036	15	25	0.040	20	30															
90	0.037	15	25	R_{min} =																	
80	0.038	15	25																		
70	0.039	15	25																		
60	0.040	15	25																		
	R_{min} = 60																				

Key:

R = Radius of curve, m
V = Ramp design speed, km/h
e = Superelevation rate
L = Length of superelevation runoff from remove crown to full super., m
A = L for 1-way ramp rotated about the centerline of travelway, m
B = L for 2-way ramp rotated about the centerline of travelway, m
NC = Normal crown
RC = Remove (adverse) crown

* For a curve radius which is between the listed values, use a straight-line interpolation to determine the superelevation rate.

RATE OF SUPERELEVATION FOR INTERCHANGE RAMPS, e_{max} = 4%
Figure 48-5D



FREEWAY LANE DROP
(Typical Schematic)

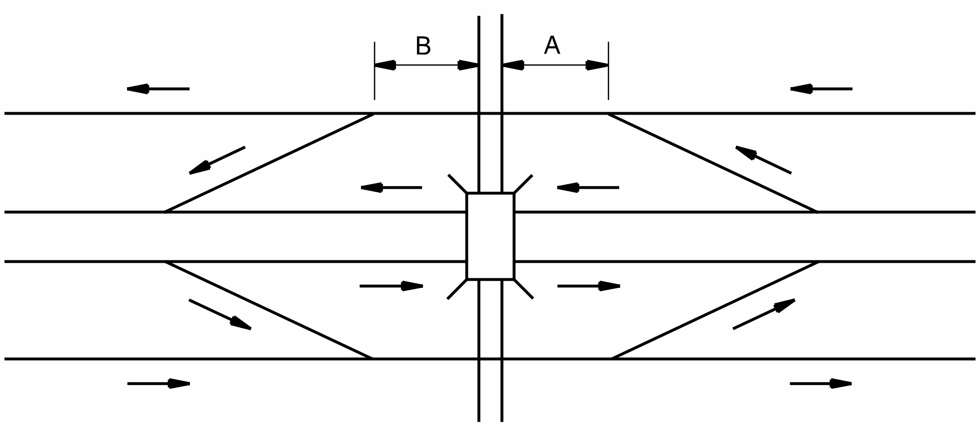
Figure 48-6A

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

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

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

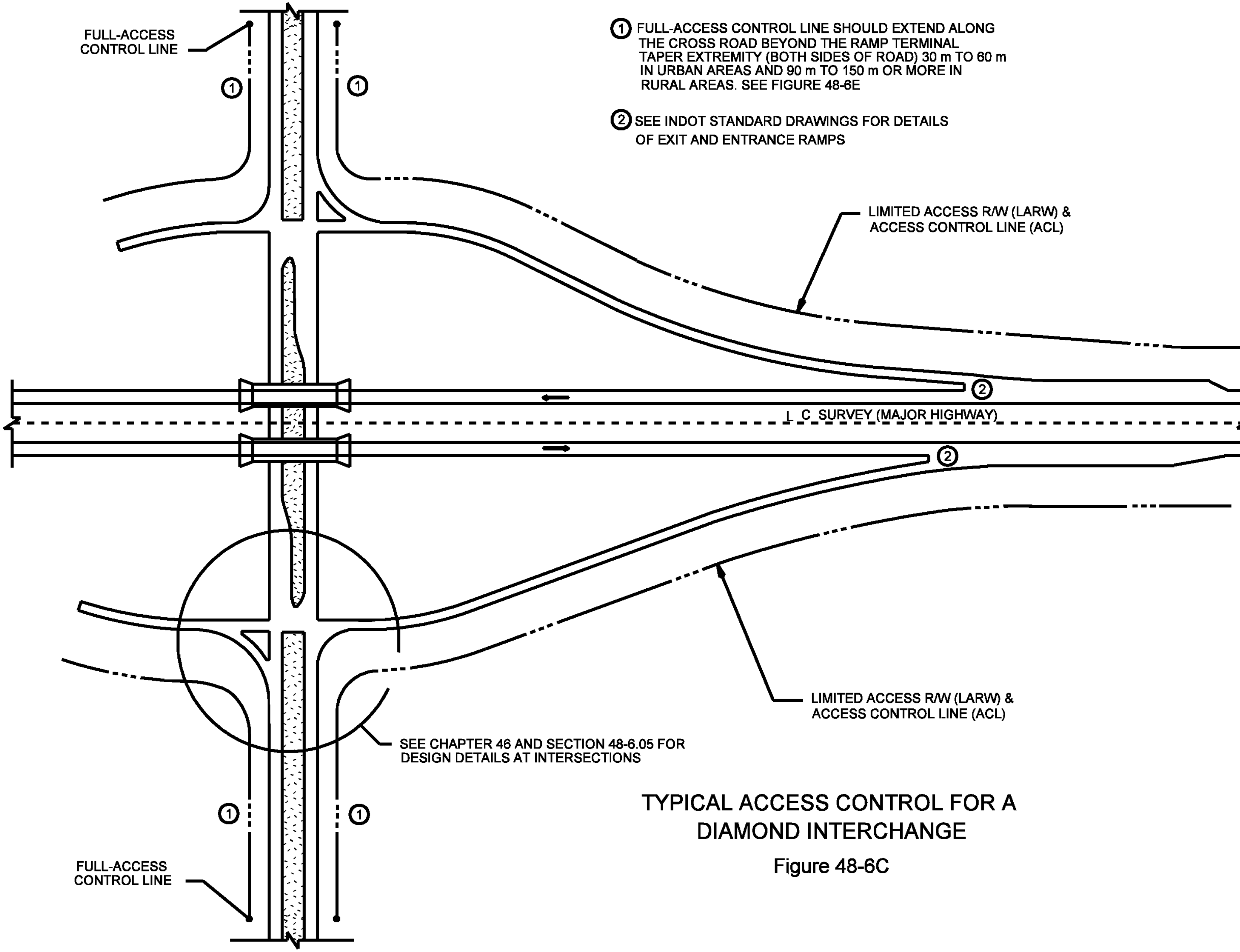
Note: Table values are acceptable for planning purposes; final dimensions will be based on a detailed operational analysis. This design may only be used where necessary in restricted urban areas.



RAMP/CONTINUOUS FRONTAGE ROAD INTERSECTION

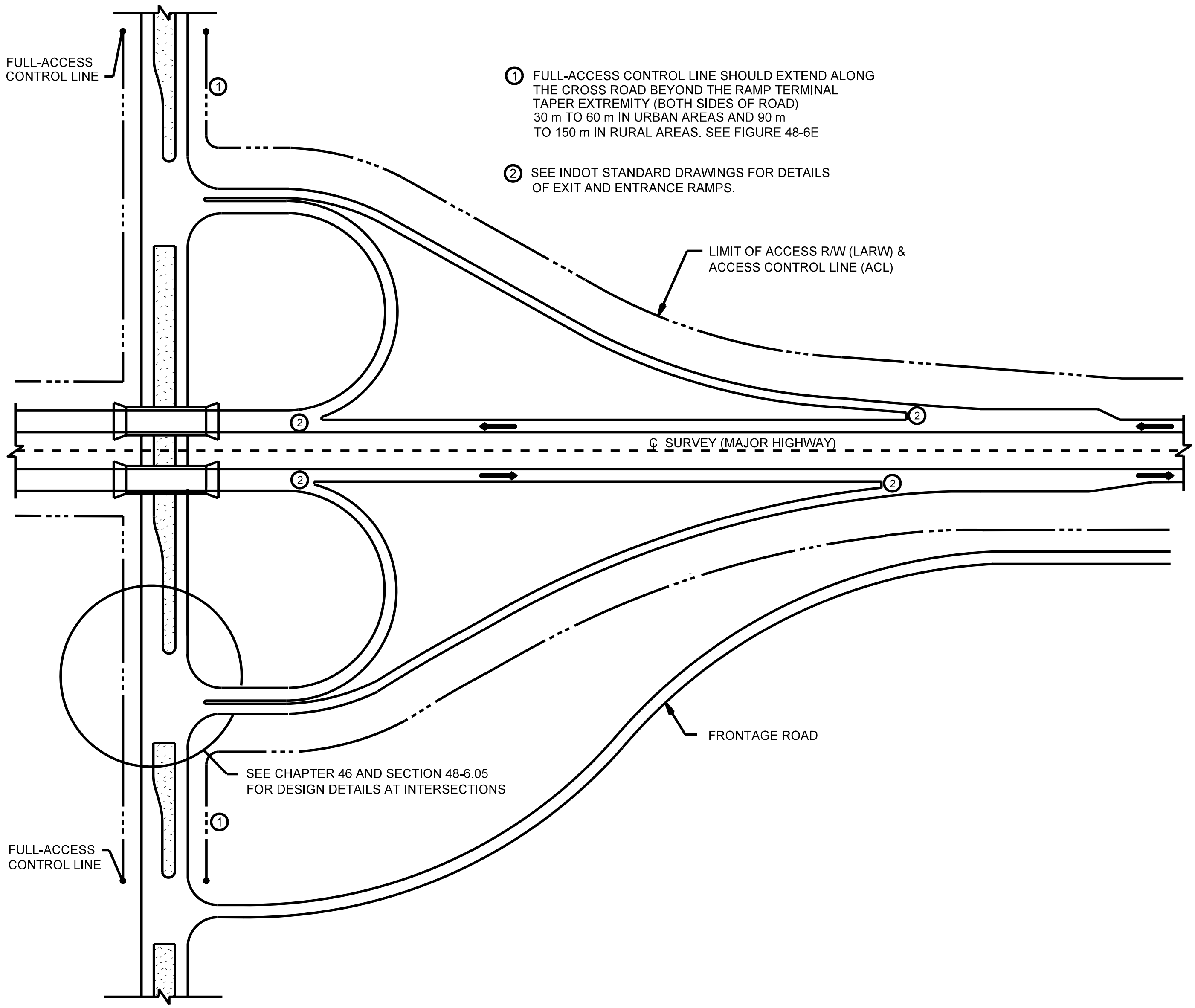
Figure B

RAMP/CONTINUOUS FRONTAGE ROAD INTERSECTION
Figure 48-6B



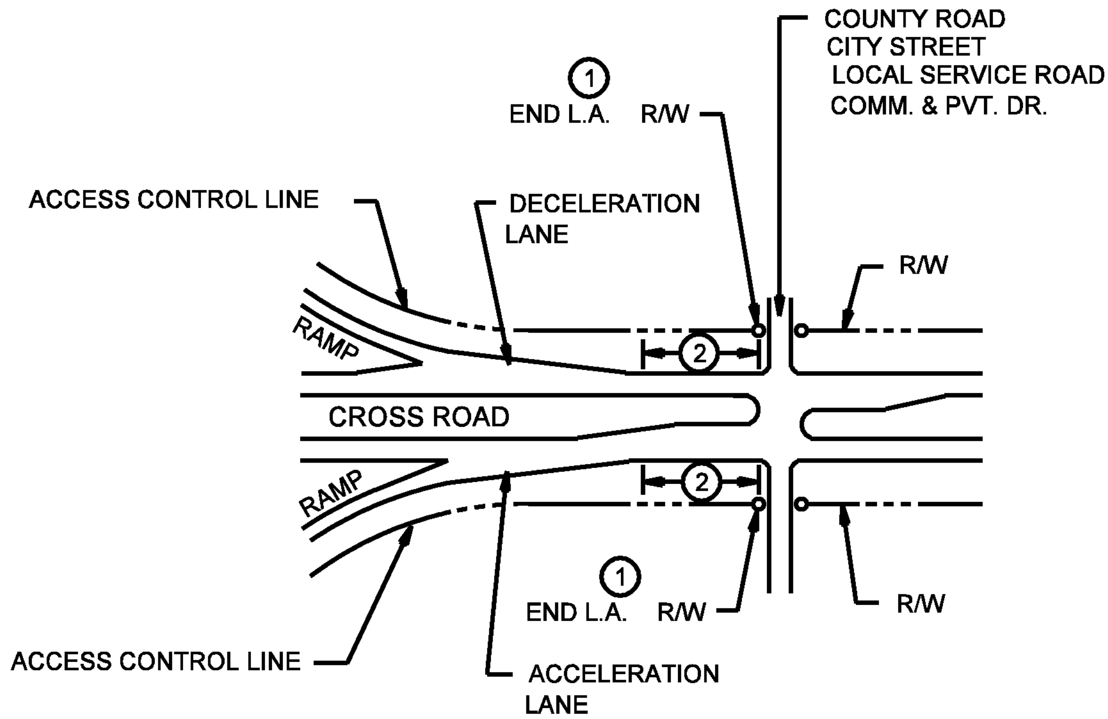
TYPICAL ACCESS CONTROL FOR A DIAMOND INTERCHANGE

Figure 48-6C



TYPICAL ACCESS CONTROL FOR A PARTIAL CLOVERLEAF INTERCHANGE
(With Frontage Road)

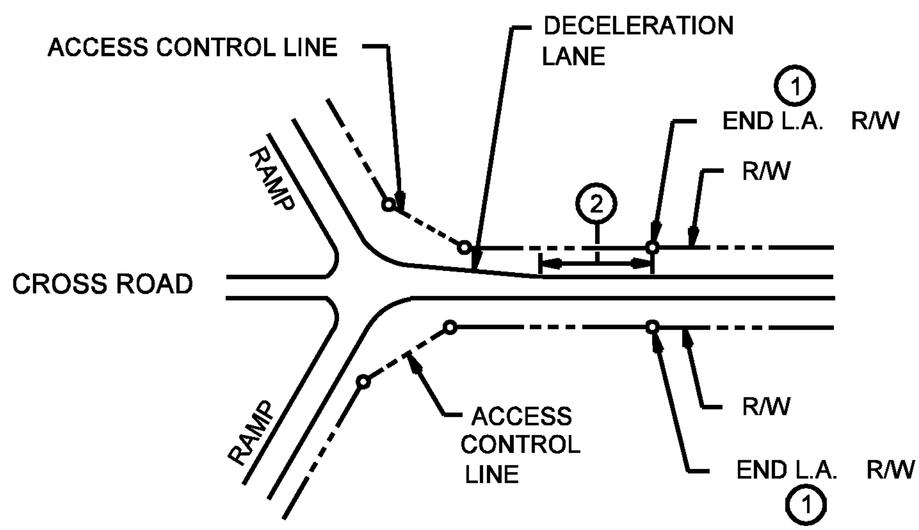
Figure 48-6D



MULTI-LANE ROADWAY
CASE I

(1) FULL-ACCESS CONTROL LINE SHOULD EXTEND ALONG THE CROSS ROAD BEYOND THE RAMP TERMINAL TAPER EXTREMITY (BOTH SIDES OF ROAD) 30 m TO 60 m IN URBAN AREAS AND 90 m TO 150 m IN RURAL AREAS. THE END OF ACCESS CONTROL SHOULD BE AT OPPOSITE POINTS, WHERE FEASIBLE.

(2) IN CASE I and II THE AUXILIARY LANE TERMINATING THE GREATER DISTANCE FROM THE INTERCHANGE AREA SHOULD GOVERN.



TWO-LANE ROADWAY
CASE II

ACCESS CONTROL AT RAMP TERMINALS

Figure 48-6E